A STUDY ON IMPROVEMENT OF UNSUITABLE SOILS FOR ROAD CONSTRUCTION USING LIME AND FLY ASH

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Degree of Master of Science

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DECLARATION

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ABSTRACT

In Sri Lanka, residual soils are abundantly encountered in constructions where residual soils with poor engineering properties have been left unused in many situations. However, it is uneconomical and non-ecological to leave these soils unused. As a solution, stabilizing the soils with chemical techniques such as lime and fly ash stabilization to the required properties has become popular and sustainable.

In this study, an unsuitable residual soil was stabilized with 3%, 5% and 8% of lime, 6%, 12% and 18% of fly ash and with 3% lime increasing the fly ash percentage at 6%, 12% and 18% by dry soil mass to investigate the variation in soil properties to be used in road constructions. First, the basic soil properties were investigated. Then the variation in plasticity characteristics, maximum dry density (MDD), optimum moisture content (OMC), unconfined compressive strength (UCS) and California bearing ratio (CBR) was studied.

A decrease in plasticity index (PI) was observed with 3% lime, but with further addition no variation observed. For soil stabilized with fly ash and lime-fly ash admixture no such variation in PI observed. There was a decrease in MDD with the increase in lime and lime-fly ash admixture percentages, but there was a slightly increasing trend with fly ash. No significant variation in optimum moisture content observed with any of the additives. An increase in the UCS of the soil with the curing time and additive percentage was observed. A significant improvement in CBR was observed with lime and lime-fly ash admixtures, but not with fly ash alone. It can be concluded that with 6% lime or 3% lime with 18% fly ash, the required CBR value for the soil to be used as a capping layer material can be achieved.

Keywords: stabilization, lime, fly ash, soil

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LIST OF ABBREVIATIONS

Abbrevation	Description
CAH	Calcium aluminate hydrate
CBR	California bearing ratio
CSH	Calcium silicate hydrate
DDL	Difused doule layer
EDS	Energy dispersive spectrometry
ICL	Initial consumtion of lime
LL	Liquid limit
MDD	Maximum dry density
OMC	Optimum moisture content
PI	Plasticity index
PL	Plastic limit
SEM	Scanning electron microscope
UCS	Unconfined compressive strength
XRD	X-ray diffraction

1. INTRODUCTION

During the early days, Engineers had the liberty to avoid any unsuitable sites or construction materials when the expected conditions for the constructions were not satisfied. As the time passed, people began to consider the economic, environmental and other factors, which in turn made it difficult to find suitable sites or suitable materials for constructions. It is a known fact that the structures such as dams, embankments, highways and airport runways require soils with good engineering properties such as low plasticity, high bearing capacity, low settlement etc. (Ansary, Noor, & Islam, 2007). With the limited resources, increase in population and rapid advancement in constructions engineers began to search for methods to improve and utilize resources, which are not generally suitable to be used in engineering applications. Therefore, it would be beneficial if weak unsuitable soils could be improved to be used in constructions.

According to (Pandey & Rabbani, 2017) the primary methods of soil stabilization are mechanical stabilisation and chemical stabilisation. In road constructions, weak subgrades are a critical issue, which results in short service life due to permanent deformation in pavements. As a solution, most of the time design engineers use methods such as removing the top layer of subgrade and replacing with good fill materials, increasing the base layer thickness and using reinforcements. Replacing these unsuitable soils with good quality materials is becoming more and more uneconomical and non-ecological. When waste generated during the removal of unsuitable soils are left unused and disposed improperly, many adverse effects such as poor hydrological conditions has been observed in the recent past. Therefore, chemical stabilization of the unsuitable soils is popular among engineers.

Selection of stabilizer for a certain soil depends on the type of soil, type of construction and the availability of the material (Derucher & Korfiatis, 1988). Different researches have been carried out using different materials such as lime, cement, fly ash, polymers and fibres to stabilize soils with better characteristics and to be environment-friendly. The use of lime for stabilization of soils has been studied by many researches such as Croft, 1964, Bell, 1996, Osinubi, 1998 and Eades & Grim, 1966. Furthermore, the use of fly ash as the sole additive and fly ash with lime has been studied with different percentages (Ansary, Noor and Islam, 2007, Kumar and Sharma, 2004, Croft, 1964a, Goecker *et al.*, 1956, Nettleton, 1963, Dissanayake, Senanayake and Nasvi, 2017 and Nawagamuwa and Prasad, 2017).

History of soil stabilization extends up to many centuries. Lime obtained from decomposing limestones and seashells at high temperatures was the cementitious material that was familiar to man before cement. There are evidence that ancient civilizations such as Greeks, Egyptians, Romans, Persians, Indians and Chinese have used lime (Arabi, 1986). According to (McDowell, 1959) stabilized roads were used in Mesopotamia and Egypt, and soil-lime mixtures were used by the Romans. With the expansion of roads and to allow for the growth in motor traffic use of soil stabilized roads began to increase (Bell, 1996). After the Second World War, there is an extensive use of stabilization on roads and runaways with the advancement of technology, laboratory experimental facilities and sophisticated analytical techniques (Arabi, 1986).

Fly ash is a by-product from burning coal in thermal power plants. The use of fly ash in concrete has been studied since the early 1930's. Laboratory experiments have proved that with partial replacement of fly ash with Portland cement can be used to produce better and low-cost concrete (Moh, Goecker, Chu, & Davidson, 1955). Fly ash lowers the heat of hydration, increase the ultimate strength and improves the workability of concrete (Davis, Carlson, & Kelly, 1937). In road construction fly ash has been used with Portland cement concrete and bituminous concrete roads (Chilcote, 1952).

The use of lime and fly ash as a stabilizing agent came into use in the 1950's. In 1951 Havelin and Khan two engineers of electric company discovered that with the addition of small amounts of hydrated lime to fly ash in the presence of water, aggregates such as sand a high compressive strength product was produced when aged for a period of 28 days or longer and a patent was granted on the use of lime and fly ash with fine aggregates (Minnick, Carson, & Mlller, 1950 and Havelin & Frank, 1951).

There are four mechanisms involved in stabilization of calcium-based stabilizers namely cation exchange, flocculation, carbonation and pozzolanic reactions (Bell, 1996 and Croft, 1964b). Cation exchange and flocculation reactions are short term while the pozzolanic reaction is a long-term reaction. Cation exchange will reduce the diffused double layer thickness and results in flocculation, reducing plasticity index with improvement in workability. Due to the pozzolanic reactions, cementitious compounds are formed, improving the strength of the soil (Cherian & Arnepalli, 2015).

In this study, the improvement of a potentially unsuitable residual soil with lime and fly ash as additives to be used in road pavement structures has been studied. The soil obtained was left unused in Central Expressway Project, Sri Lanka because of the strength and compaction characteristics of the soil. The objective of this study is to measure the optimum additive percentages, for the improvement of the soil to the required specifications.

2. LITERATURE REVIEW

2.1.Residual Soils

Residual soils are different from transported soils. Residual soils are formed by chemical or physical weathering of rock underlying them. According to (A. L. Little, 1969) weathered products can be classified into 6 scales of weathering, namely fresh rock (Grade I), slightly weathered rock (Grade II), moderately weathered rock (Grade III), highly weathered rock (Grade IV), completely weathered (Grade V) and residual soil (Grade VI). In residual soils, all the rock materials are converted to soil. A large change in volume can be observed although the soil has not been transported (Fookes, 1997).

Residual soil properties highly vary according to the degree of weathering. Weathering can be from physical, chemical and biological means (Blight & Leong, 2012). Physical processes include stress release by erosion, differential thermal strain and ice and salt crystallization pressure, which will comminute the rock, increase the permeability exposing the fresh surface to chemical attack. Chemical processes such as hydrolysis, cation exchange and oxidation alter the rock minerals to form clay minerals (Mitchell, 1976). Biological weathering includes splitting by roots, bacteriological oxidation, chelation, etc.

Weathering is controlled by climatic conditions and topographical conditions. Physical weathering is dominant in dry climates and chemical weathering rate is controlled by the availability of moisture and temperature (Blight & Leong, 2012). Sri Lanka receives an annual rainfall of about 900 mm in the driest parts and about 5000 mm in the wettest parts. This soil was obtained from Mirigama area in Sri Lanka where the annual rainfall is about 2000 mm and the average temperature is about 27^0 C. With regards to the climatic factors Duchafour (1982), classified different phases in residual soils as shown in Figure 1. According to Duchafour (1982), ferrallitic soils form in hot, humid tropics where the annual rainfall is greater than 1500mm and mean temperature is above 25^0 C which is satisfied in Mirigama area. According to Uehara (1982), around equator due to high temperature and rainfall throughout the year results in formation of low active kaolinite and oxides.

Phase	Soil type	Zone	Mean annual temperature (°C)	Annual rainfall (m)	Dry season
1	fersiallitic	mediterranean, subtropical	13–20	0.5–1.0	yes
2	ferruginous ferrisols (transitional)	subtropical	20–25	1.0–1.5	sometimes
3	ferrallitic	tropical	>25	>1.5	no

Figure 1: Summary of residual soil phases with regards to climatic conditions according to Duchafour (1982). Source: (Fookes, 1997)

2.2.Stabilization

Soil stabilization or improvement is alteration of soil to enhance its properties. Stabilization can be achieved by means of thermal, electrical, mechanical and chemical methods (Little & Nair, 2009). Thermal and electrical stabilization of soils are not often used, as these methods are expensive. Mechanical stabilization includes stabilization by densification of soil using compaction and vibration. With the expulsion of air from soil without changing the water content densification occurs. Chemical stabilization is alteration of physio-chemical properties of soils using different additives such as cement, lime, fly ash, polymers, fibres, cement kiln dust etc.

2.2.1. Chemical stabilization

According to Petry and Little (2002), the chemical stabilizers can be classified as traditional stabilizers, non-traditional stabilizers and by-products. Some examples of different chemical stabilizers used grouped into these three categories as shown in Table 1.

Table 1: Classification of different stabilizers (Petry and Little, 2002).

Traditional Stabilizers	Non-traditional stabilizers	By-products
Lime	Sulfonated Oils	Cement kiln dust
Portland cement	Ammonium chloride	Lime kiln dust
Fly ash	Enzymes	
	Polymers	
	Potassium compounds	

2.2.2. Guidelines for stabilization of soils

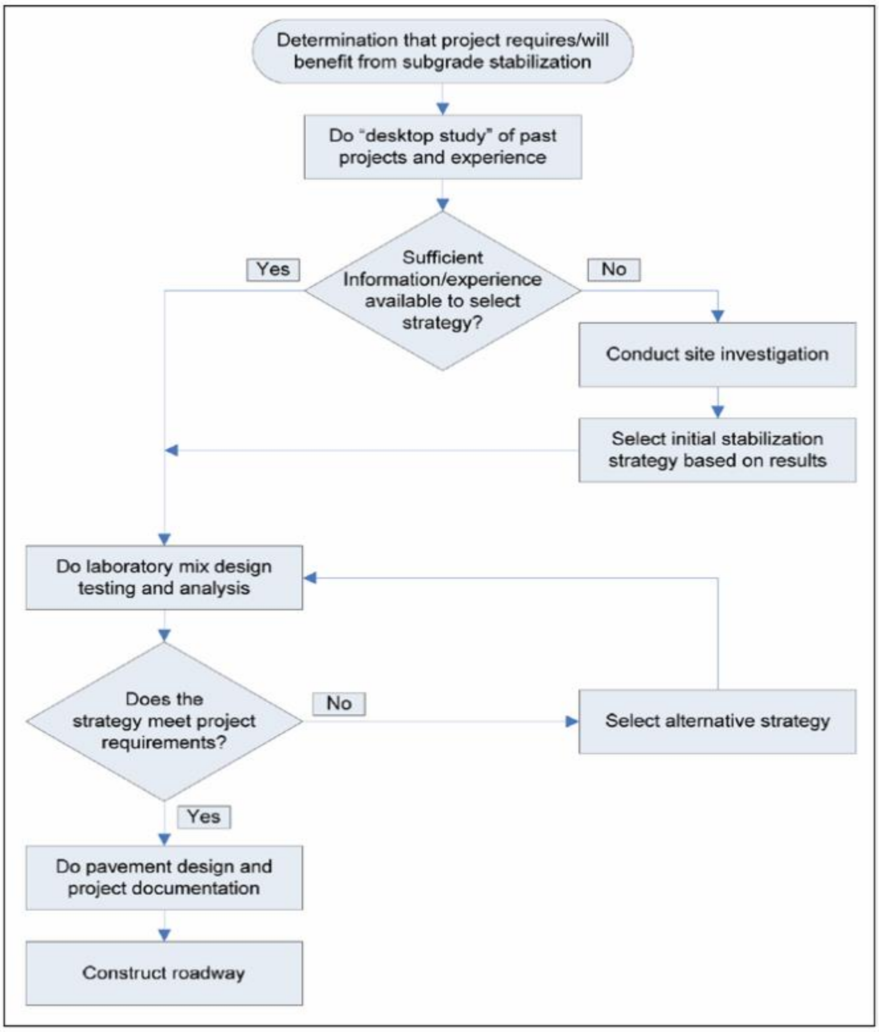


Figure 2: Overview of stabilization design procedure (Jones, Rahim, Saadeh, & Harvey, 2012).

Guidelines for stabilization of soil have been developed by different institutions such as Texas Department of Transportation (TxDOT), the US Army and Air Force, Portland Cement Association (PCA). National Lime Association (NLA), Federal Highway Association and other relevant agencies throughout the world. According to (Jones et al., 2012) chemical stabilization of subgrade soils depends on the type of soil, drainage, type and amount of stabilizer, construction methods followed. A more generalized flow chart on the process involved in chemical stabilization is presented in Figure 2. Figure 3 presents a flowchart from Texas Department of Transportation (2005), guidelines for subgrade stabilization.

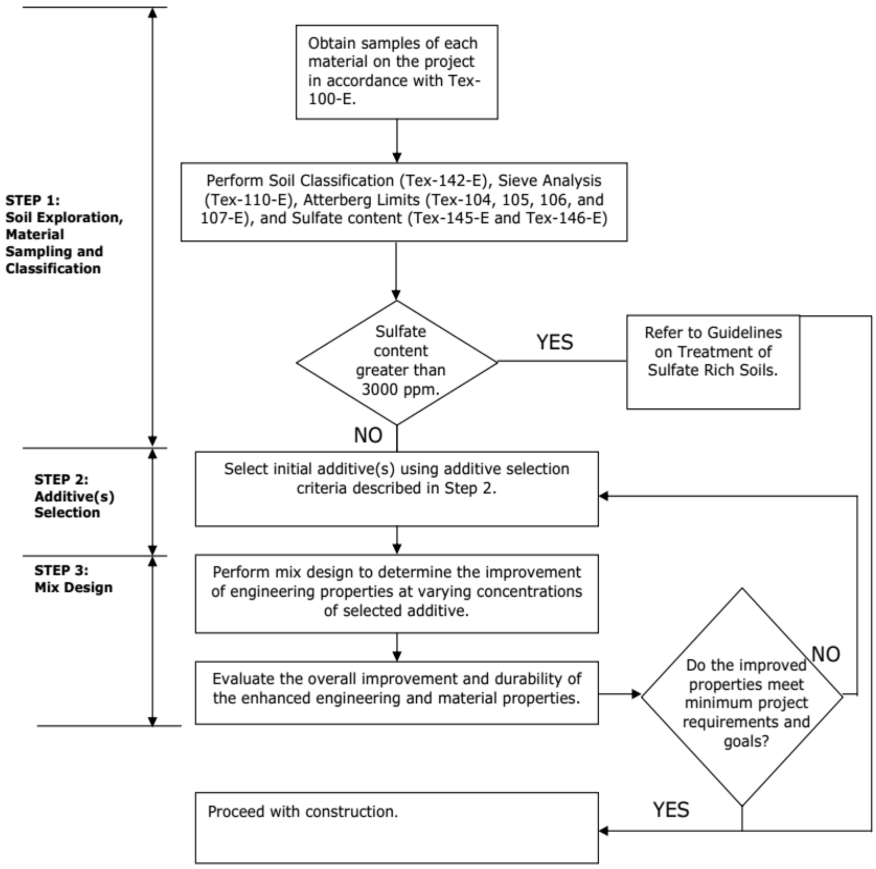


Figure 3: Flowchart for subgrade stabilization (Texas Department of Transportation, 2005).

2.2.3. Selection of the stabilizer

The selection of the stabilizer depends on the soil characteristics and the ability of a given stabilizer to improve physiochemical properties of the soil. According to Little and Nair (2009), the following should be considered when selecting the stabilizer.

1. Soil consistency and gradation
2. Soil mineralogy composition
3. Desired engineering properties
4. Purpose of treatment
5. Mechanisms of stabilization
6. Environmental conditions and engineering economics

In Figure 4, a decision tree to identify the suitable stabilizers for subgrade soils with PI and percentage passing the no. 200 sieve is given. In (U.S. Army TM 5-882-14/AFM 32-1019, 1994) stabilizer selection is based on the soil classification and plastic limits as shown in Figure 5.

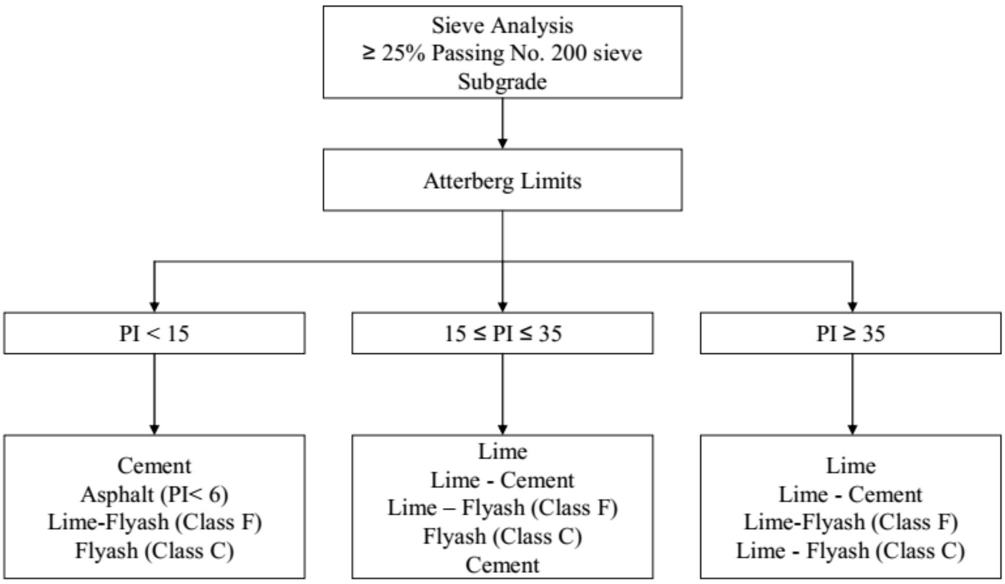


Figure 4: Decision tree for selecting stabilizers for use in subgrade soils (Texas Department of Transportation, 2005).

Area	Soil Classification	Type of Stabilizing Additive Recommended	Restriction on LL and PI of Soil	Restriction of % Passing No. 200 sieve	Remarks
1A	SW or SP	(1) Bituminous (2) Portland -Cement			
		(3) Lime-Cement-Fly Ash	PI not to exceed 25		
1B	SW-SM or SP-SM or SW-SC or SP-SC	(1) Bituminous	PI not to exceed 10		
		(2) Portland -Cement	PI not to exceed 30		
		(3) Lime	PI not to exceed 12		
		(4) Lime-Cement-Fly Ash	PI not to exceed 25		
1C	SM or SC or SM-SC	(1) Bituminous	PI not to exceed 10	Not to exceed 30% by weight	
		(2) Portland -Cement	*		
		(3) Lime	PI not less than 12		
		(4) Lime-Cement-Fly Ash	PI not to exceed 25		
2A	GW or GP	(1) Bituminous			Well-graded material only
		(2) Portland -Cement			Material should contain at least 45% by weight of material passing No.4 sieve
		(3) Lime-Cement-Fly Ash	PI not to exceed 25		
2B	GW-GM or GP-GM or GW-GC or GP-GC	(1) Bituminous	PI not to exceed 10		Well-graded material only
		(2) Portland -Cement	PI not to exceed 30		Material should contain at least 45% by weight of material passing No.4 sieve
		(3) Lime	PI not less than 12		
		(4) Lime-Cement-Fly Ash	PI not to exceed 25		
2C	GM or GC or GM-GC	(1) Bituminous	PI not to exceed 10	Not to exceed 30% by weight	Well-graded material only
		(2) Portland -Cement	*		Material should contain at least 45% by weight of material passing No.4 sieve
		(3) Lime	PI not less than 12		
		(4) Lime-Cement-Fly Ash	PI not to exceed 25		
3	GH or CL or MH or ML or OH or OL or ML-CL	(1) Portland	LL less than 40 and PI less than 20		Organic and strongly acid soils falling within this area are not susceptible to stabilization by ordinary means
		(2) Lime	PI not less than 12		

* PI ≤ 20 + [(50-percent passing No. 200 sieve) / 4]

Figure 5: Guide for selecting additives (U.S. Army TM 5-882-14/AFM 32-1019, 1994).

2.2.4. Lime

Lime is one of the oldest and most commonly used stabilizers known to man. Lime is obtained from burning limestone or dolomite at high temperatures. The common final products of production of lime are quick lime or hydrated lime (Bhengu &

Allopi, 2016). The reaction involved in manufacturing quick lime from limestone is shown in Equation 1. CO_2 is produced as a by-product of this reaction. Transformation of quick lime to hydrated lime follows a hydration reaction as in Equation 2.

$CaCO_3 + \text{Heat} \rightarrow \text{CaO} + CO_2$ ……………………………………………..Equation 1

$\text{CaO} + H_2\text{O} \rightarrow Ca(OH)_2 + \text{Heat}$ ……………………….……………..Equation 2

2.2.4.1.Soil lime reactions

There are four main chemical reactions occurring when a soil is mixed with lime.

a. Cation exchange
b. Flocculation
c. Pozzolanic reactions
d. Carbonation

Cation exchange and flocculation reactions are short-term reactions and commonly known as “modification reactions”. Pozzolanic reactions are long-term reactions also called as “stabilization reactions”. Carbonation can occur when lime reacts with carbon dioxide.

a. Cation Exchange

Cation exchange reactions take place very quickly when the soil and lime are mixed with water (Mallela, Quintus, & Smith, 2004). To neutralize the charge deficiency in negatively charged crystal structures of clay, dipolar water molecules and cations attract to the negatively charged cleavage. This will form a diffused separation of two charged surfaces called “double layer”. Thicker double layers will result in more active and more plastic soils (Prusinski & Bhattacharja, 1999).

When hydrated lime is added to soil $Ca(OH)_2$ will dissociate into calcium ions and hydroxyl ions. Hydroxyl ions will result in an increase of pH of the soil-lime mixture. When CaO is added instead of $Ca(OH)_2$ an exothermic reaction will occur as in Equation 2 reducing the water content of the soil mix. Then of $Ca(OH)_2$ will dissociate following the reaction in Equation 3.

$Ca(OH)_2 \rightarrow Ca^{2+} + 2OH^-$ .... Equation 3

The increase in pH is favourable for the exchange of divalent Ca^{2+} ions with monovalent ions (Na^+, K^+, etc.) present in the diffused double layer (DDL) of negatively charged soil minerals such as (Mitchell & Soga, 2005). Higher valence cations replace lower valence cations and smaller cations are replaced by larger cations. Replacement of cations takes place in order of the replacing power (Ravina & Gurovich, 1977).

$Li^+ < Na^+ < H^+ < NH_4^+ < Mg_2^+ < Ca^{2+} < Al^{3+}$

The exchange of di or tri valent ions will result significant reduction in DDL thickness. Due to the increase in pH and the electrolyte concentration reactive silica (Si^{4+}) ions and alumina (Al^{3+}) ions present in soil minerals dissolute to pore solution (Bell, 1996).

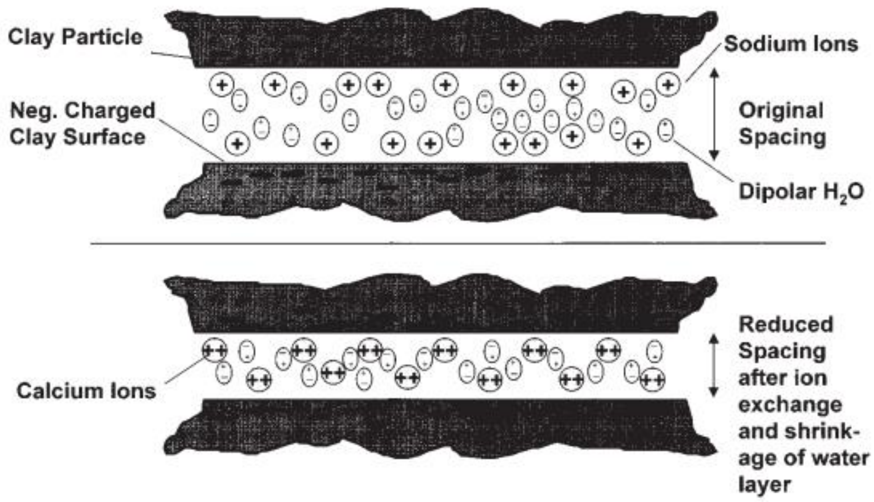


Figure 6: Cation exchange (Prusinski & Bhattacharja, 1999).

b. Flocculation

Flocculation and agglomeration of clay particles will result in a change of texture due to the formation of large particles. Flocculation is the process of clay particles altering their flat parallel structure to more random edge to face orientation as shown in Figure 7 (Herzog & Mitchell, 1963). With the reduction of DDL thickness, flocculation and agglomeration, plasticity of the soil drops while shear strength and workability increases.

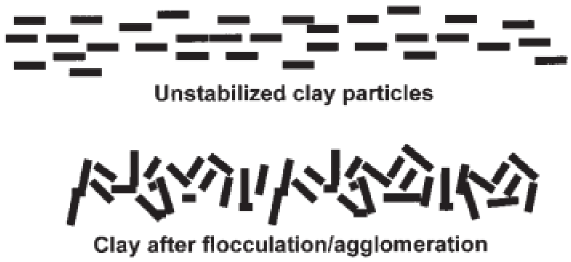


Figure 7: Flocculation and agglomeration (Prusinski & Bhattacharja, 1999).

c. Pozzolanic reactions

Pozzolanic reactions are slow reactions which take place over months and years. Pozzolanic reactions result in an increase of strength, reduction of plasticity and increase in gradation. When lime is added to soils, first the affinity of the soil for lime must be satisfied. Otherwise, lime will not be available for pozzolanic reactions. This process has been referred to as lime fixation. To identify the lime fixation point the optimum lime content for maximum increase in plastic limit can be used. Further addition of lime will result in an increase in strength (Hilt & Davidson, 1960b).

With adequate lime, the pH of the soil-lime mixture increases to 12.4 which is the pH of saturated lime solution (Eades & Grim, 1966). This high pH increases the solubility and reactivity of silica and alumina. Calcium ions combine with reactive silica and alumina to form calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH) gels. These gels have the ability to bind clay particles together and with time, the gel crystallises into cementitious compounds (Bell, 1996). The reactions for the formation of cementitious products are shown in Equation 4 and Equation 5.

$$Ca^{2+} + OH^{-} + SiO_2 \rightarrow CSH\ (Calcium\ silicate\ hydrate) \ \ldots\ldots\ldots\ldots \ \text{Equation 4}$$

$$Ca^{2+} + OH^{-} + Al_2O_3 \rightarrow CAH\ (Calcium\ aluminate\ hydrate) \ \ldots\ldots \ \text{Equation 5}$$

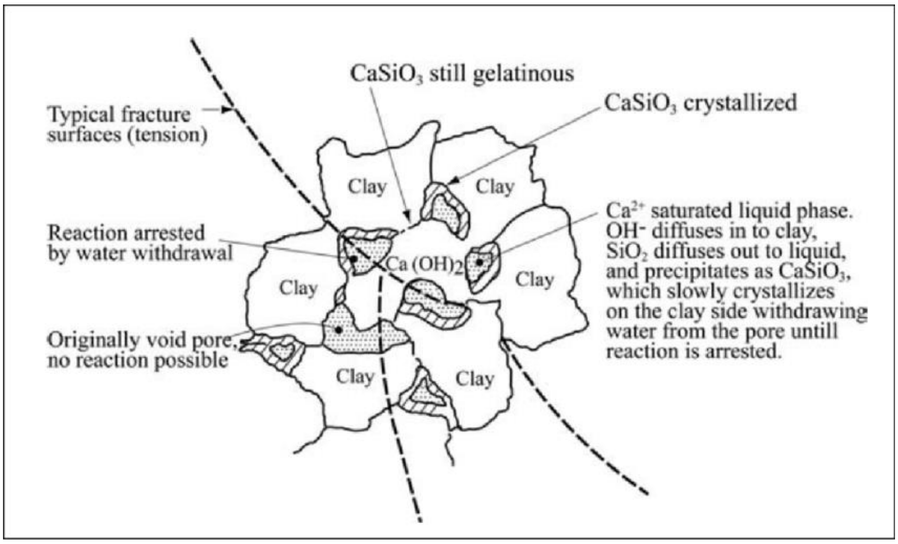


Figure 8: Mechanism of lime stabilization (Ingles & Metcalf, 1973)

Different cementitious compounds are formed for different soils and these reaction products take different times to form. For montmorillonite clay, which is expansive shows early strength gain and produces reaction products calcium aluminium hydrate in the form of C_4AH_{13} and calcium silicate hydrate in the form of CSH (Croft, 1964b). When Kaolinite is treated with lime, calcium silicate hydrates in the form of CSH and $C_3S_2H_3$, and calcium aluminate hydrate in the forms of C_4AH_{13}, CAH_{10} and C_3AH_{11} (Bell, 1996) formed. According to Croft (1964), illite was slow to react and no aluminates observed, but CSH resulted in long-term strength.

Elevated temperatures and long curing periods are favourable factors for pozzolanic reactions. A significant improvement in strength was observed with the increase in temperature and when the temperature drops below 4^0 C the pozzolanic reactions are retarded (Bell, 1996).

d. Carbonation

Carbonation of lime is an undesirable reaction that occurs in lime stabilized soils (Croft, 1964b). CO_2 in the air dissolves with pore water and reacts with hydroxyl ions forming carbonates. Then the formed carbonates react with calcium ion to form $CaCO_3$, which is a weak cement. Carbonation reactions are given in Equation 6 and Equation 7.

$CO_2 + 2OH^- \rightarrow CO_3^{2-} + H_2O$ …………………….……………….. Equation 6

$CO_3^{2-} + Ca^{2+} \rightarrow CaCO_3$ …………………………………………..... Equation 7

2.2.4.2.Optimum lime content

According to Bell (1996), optimum lime content (OLC) or the amount of lime required for modification and stabilization in soils depends on the soil type, desired improvement, type of lime, environment conditions, etc. First, the affinity for Ca^{2+} ions owing to charge deficiency is satisfied when lime is added to soil. The amount of lime required for the fixation of lime in soil is known as lime fixation point or initial consumption of lime (ICL) and without satisfying the affinity, lime is not available for other reactions (Hilt & Davidson, 1960). This lime fixation point could be identified by finding the lime percentage, for maximum increase in plastic limit of the soil. Bell (1996), found that 1-3% lime could bring about the changes in plasticity. According to Hilt and Davidson (1960), the ICL can be mathematically calculated using a relationship between ICL and clay size fraction, as shown in Equation 8.

$$ICL = \frac{Clay\ size\ fraction\ (\%)}{35} + 1.25$$ ………………………….………. Equation 8

ICL does not take part in pozzolanic reactions, as there are no free Ca^{2+} ions available for long-term reactions. Pozzolanic reactions can occur if provided with sufficient reactive clay minerals, free calcium and moisture. Eades and Grim (1966), introduced the concept of OLC. OLC is the amount of lime required, for short-term improvement of soil in terms of workability and plasticity, and long-term improvement of the strength (Little, 1987). A thumb rule to identify the percentage of OLC is adding 1% lime for every 10% of clay fraction present in the soil (Ingles and Metcalf, 1973). The clay fraction usually will not exceed 80%. Therefore, the OLC will be below 8%. Basma and Tuncer (1991), reported that soil stabilization using lime can be brought by 2- 8% lime.

2.2.5. Fly ash

Fly ash is a by-product formed during power generation using powdered coal and transported with flue gas. Combustion of coal produce ash and about 80% of this ash is fly ash (Sivapullaiah, Prashanth, Sridharan, & Narayana, 1998). Fly ash is made up of tiny spheres of silica and alumina glass. Fly ash is a pozzolanic material similar to volcanic ash. Pozzolans are siliceous or aluminous inert materials on their own, but reacts with lime and water to form compounds having cementitious properties (Moh et al., 1955). Fly ash is one of the most common artificial pozzolan known to humans.

In Sri Lanka, Norochchole Lakvijaya thermal power plant produces about 200, 000 metric tons of fly ash in generating power annually. Fly ash generated is dumped in dump yards 100 m away from the plant. Out of this, only a small portion is used in cement production (Dissanayake et al., 2017). As this area is closer to the coast, the wind speed is very high. This has resulted in dust emission to the nearby settlements.

According to ASTM C618 (2004), fly ash can be classified as class C and class F fly ash. Class C is produced from lignite or subbituminous coal burning. Class C fly ash has the pozzolanic properties as well as cementitious properties. Class C fly ash consists of calcium combined with silica or alumina where hydration reactions occur with water. Some free limes occur in these hydration reactions where this free lime can react with reactive silica or alumina from clay or fly ash (Little & Nair, 2009).

Class F fly ash is produced from burning anthracite or bituminous coal. Class F fly ash has only pozzolanic properties with low concentration of calcium. Therefore, class F fly ash requires an activator such as lime or cement (Little & Nair, 2009). The chemical composition for class C and class F classification of fly ash is given in Table 2.

Table 2: Chemical requirement (ASTM C618, 2004)

	Class C	Class F
SiO_2 + Al_2O_3 + Fe_2O_3, min%	50.0	70.0
SO_2, max%	5.0	5.0
Moisture content, max %	3.0	3.0
Loss on ignition, max %	6.0	6.0

Similar to lime when fly ash is added to short-term reactions as well as long-term reactions occur. Short-term reactions include cation exchange, flocculation and agglomeration, and long-term reactions involve pozzolanic reactions. Lime (CaO) in fly ash quickly reacts with water to produce calcium (Ca^{2+}) ions which replace the monovalent cations in clay minerals (Centiner, 2004). Long-term pozzolanic reactions are due to the reaction between reactive silica in fly ash and free lime which produce calcium silicate hydrates (CSH) as shown in Equation 9. The pozzolanic reactivity of fly ash depends on the amount of reactive silica in fly ash, free lime and moisture, fineness of fly ash and low carbon content (Sivapullaiah et al., 1998).

$$Ca^{2+} + OH^{-} + SiO_2\ (Fly\ ash\ silica) \rightarrow CSH \quad \text{......Equation 9}$$

2.2.6. Stabilization procedure and quality control in use, according to U.S. Army TM 5-882-14/AFM 32-1019 (1994).

Stabilizing with lime, fly ash and lime-fly ash mix follows somewhat similar procedure. Methods of stabilization are in-place mixing, plant mixing and pressure injection. In place mixing can be carried out in three methods.

1. Mixing additives with the soil in the construction sites
2. Mixing off-site with borrow soil, transport it and compact
3. Borrow soil is hauled to the site, mixed and compacted.

In plant mixing operation the soil is brought to a plant and soil is mixed with predetermined additive content and water. Then the soil is transported to the site and compacted. Pressure injection involves injecting lime slurry to a depth of about 7 to 10 feet.

According to U.S. Army TM 5-882-14/AFM 32-1019 (1994), construction steps are as follows.

1. Soil preparation
2. Additive application
3. Pulverization and mixing
4. Compaction
5. Curing

In soil preparation, the subgrade soil should be brought to final grade and alignment. Trimming can be used to bring the subgrade elevation to the required level. Additives can be added to the soil in dry or slurry form. Then to obtain satisfactory soil additive mixtures pulverization and mixing should be done. For heavy clays, two-stage pulverization and mixing should be used and for other soils one stage mixing is satisfactory. In two-stage mixing for preliminary mixing disc harrows and grader scarifiers can be used and for final mixing one pass travel plant mixers or high-speed rotary mixers are required. For one-stage mixing blade mixers and rotary mixers can be used, but rotary mixers are preferred for a more uniform mix.

To achieve a good strength and durability soil must be compacted. When compacting, most guidelines specify to achieve 90 or 95 percent of ASTM D 698 or ASTM D 1557 maximum dry density. To achieve high densities, soil should be compacted to optimum moisture content with appropriate compacters are necessary. For granular soils, immediate compaction or delays up to 2 days are not detrimental and for fine-grained soil, immediate compaction or delays up to 4 days are not detrimental.

For a maximum strength and durability, soil must be cured properly. Favourable curing conditions such as favourable temperatures (40-50 ^{0}C) and moisture content (around optimum moisture content) should be provided for a passage of time.

To control the quality of lime stabilized soils most important factors that should be controlled are pulverization and scarification, lime content, uniformity of mixing, time sequence of operations, compaction and curing. Before adding additives for stabilization pulverization and scarification of soil is required. To assure this has happened properly sieve analysis can be used. To confirm the quantity of lime slurry required to provide desired lime content checking the specific gravity of the slurry can be used. To check whether a uniform mix throughout the depth of treated soil has been achieved, a phenolphthalein indicator can be used. With the presence of free lime, the soil will turn into a reddish pink colour.

Proper compaction can be achieved by controlling moisture-density. To determine the density of the compacted soils sand cone method, core cutter method, balloon air method and nuclear methods can be used. Curing should be carried out to achieve final desired properties so that lime will not become non-reactive due to carbonation.

2.2.7. Specifications for the road use

There are many design guides and manuals used for the road constructions and pavement design throughout the world. In Sri Lanka, design guides such as Road note 31 (Transport and Road Research Laboratory (TRRL), 1984), ICTAD guidelines (ICTAD, 2009) and Australian pavement design guidelines (Austroad, 2017) are used. The residual soil used for the experiments was left unused considering the engineering properties such as grain size distribution, compaction characteristics and strength characteristics in Central Expressway Project (CEP) package 2 from Kossinna to Mirigama, Sri Lanka. Some of the design guide requirements, for the soil to be used in road constructions are summarized in the following section.

According to (Transport and Road Research Laboratory (TRRL), 1984) three main aspects should be considered in road pavement design.

1. Amount of traffic and equivalent standard axle load
2. The strength of the subgrade soil
3. Selecting the most economical combination of pavement materials.

Components of flexible and rigid pavement structures are presented in Figure 9. In Sri Lanka usually flexible road pavements are constructed which includes surface, road base, subbase and subgrade. This research was targeted on improving the unsuitable residual soils as capping layer (selected or improved subgrade) material.

According to (Austroad, 2017) stabilization of soil can be used to increase the strength and performance of subgrade materials, optimise the use of available pavement materials and reduce the layer thickness compared to unbound materials.

The subgrade soil strength is commonly determined using the CBR value. According to Transport and Road Research Laboratory (TRRL) (1984), the top 250 mm of the subgrade should be compacted to a density of at least 100% of maximum dry density achieved from British standard light compaction or at least 93% of maximum dry density from British standard heavy compaction. There are different subgrade strength classes defined according to their CBR value in Transport and Road Research Laboratory (TRRL) (1984). The subgrade classes are as shown in Figure 10. For different traffic classes and subgrade strength classes charts have been given for economical designs of pavement materials. The chart for granular road base or surface dressing provided in (Transport and Road Research Laboratory (TRRL), 1984) is shown in Figure 11.

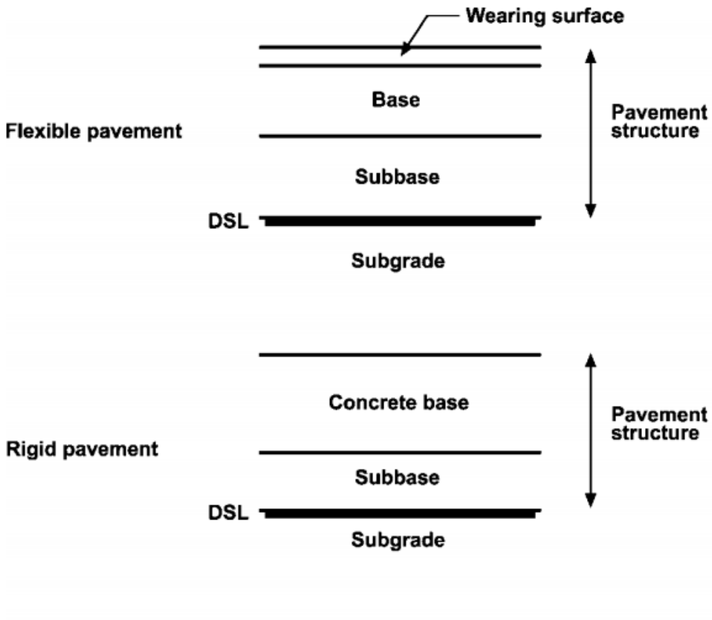
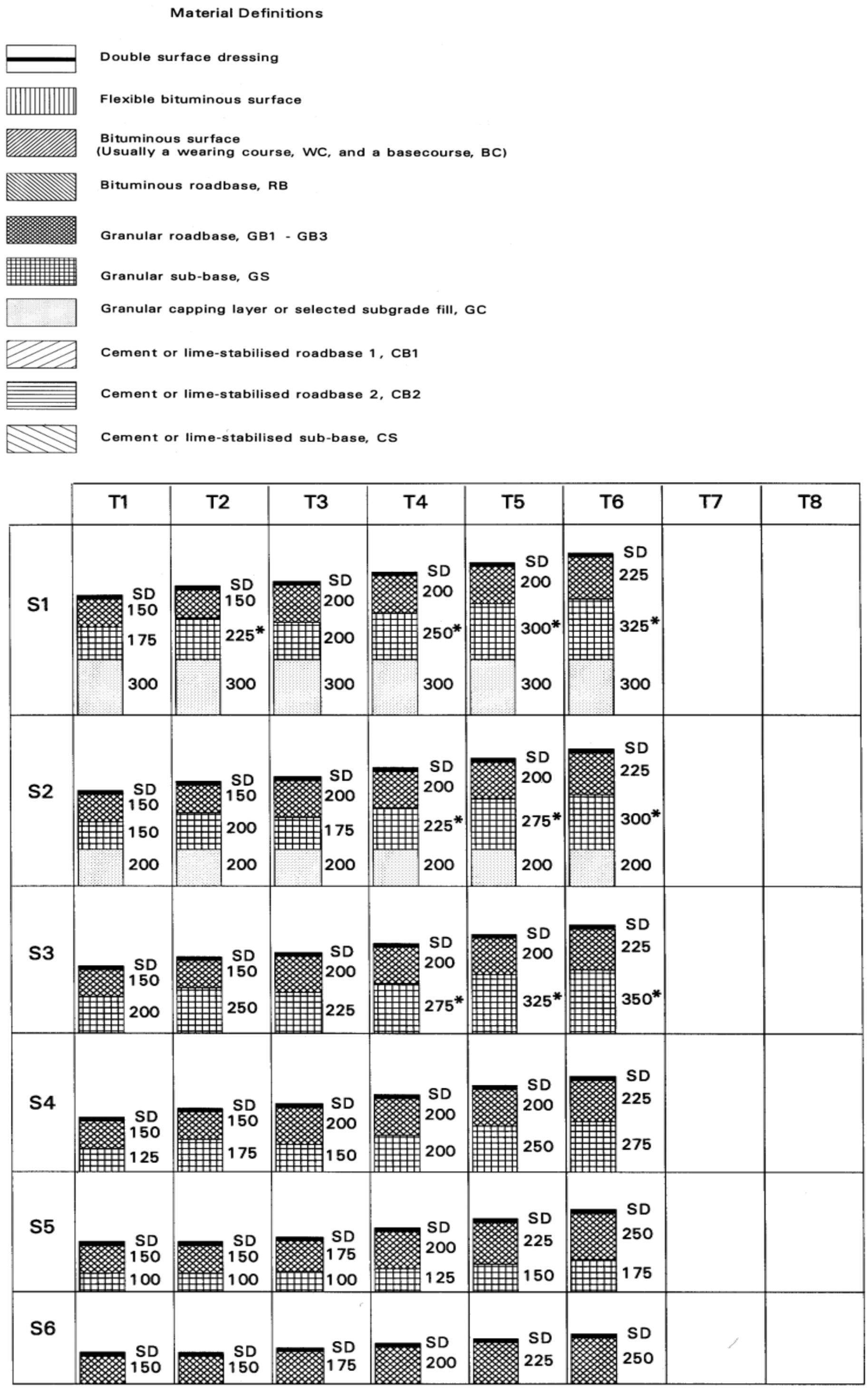


Figure 9: Components of flexible and rigid road pavement structures (Austroad, 2017).

Subgrade strength classes

Class	Range (CBR %)
S1	2
S2	3 - 4
S3	5 - 7
S4	8 - 14
S5	15 - 29
S6	30

Figure 10: Subgrade strength classes (Transport and Road Research Laboratory (TRRL), 1984).



Note: 1 * Up to 100mm of sub-base may be substituted with selected fill provided the sub-base is not reduced to less than the roadbase thickness or 200mm whichever is the greater. The substitution ratio of sub-base to selected fill is 25mm : 32mm.

2 A cement or lime-stabilised sub-base may also be used.

Figure 11: Chart 1 Granular road base/surface dressing (Transport and Road Research Laboratory (TRRL), 1984).

It can be observed that with the increase in subgrade strength the thickness of the granular road base and granular subbase can be decreased. When the subgrade strength is low then a capping layer is required. For selected subgrade or capping material, a minimum CBR value of 15 is recommended (Transport and Road Research Laboratory (TRRL) (1984), Austroad (2017) and Design Manual for Roads and Bridges (DMRB) (1995)). Formation of subgrade and capping layer is shown in Figure 12. According to the Road Development Authority (2015), the specifications for capping layer material is shown in Table 3. However, for the pavement designs, the subgrade materials should not be assigned with CBR values greater than 15% (Austroad, 2017).

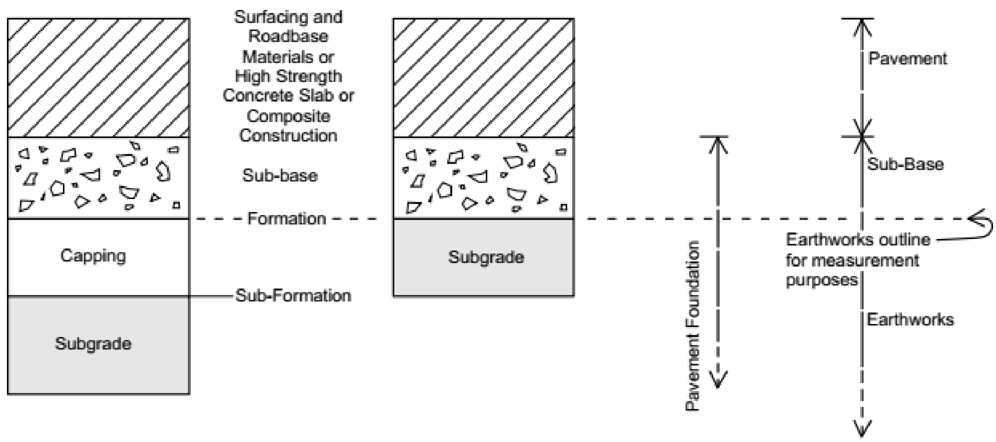


Figure 12: Formation of pavement foundation (Austroad, 2017)

Table 3: Specifications for capping layer material (Road Development Authority, 2015).

Capping layer	LL (Max)	40
	PI (Max)	15
	CBR (4 days soaked) (Min)	15

2.3.Reported geotechnical properties for soil treated with lime, fly ash and with lime-fly ash admixture.

2.3.1. Plasticity characteristics

In many researches (Bell (1996), Kumar & Sharma (2004), Dissanayake, Senanayake, & Nasvi (2017)) an immediate reduction in plasticity index (PI) of soils and an increase in the workability was observed upon addition of lime and fly ash. However, the change in liquid limit and plastic limit direction is different from one soil to another (Sivapullaiah, Sridharan, & Bhaskar Raju (2000), Dash & Hussain (2012)).

Bell (1996), studied the change in plasticity characteristics of clay minerals (Kaolinite, Montmorillonite and Quartz) with different lime percentages. The change in LL and PL observed is presented in Figure 13. As observed the LL of Kaolinite increased while in Montmorillonite and Quartz with lime, LL increased. Croft (1964), reported that the increase in LL of Kaolinite was due to the action of hydroxyl ions.

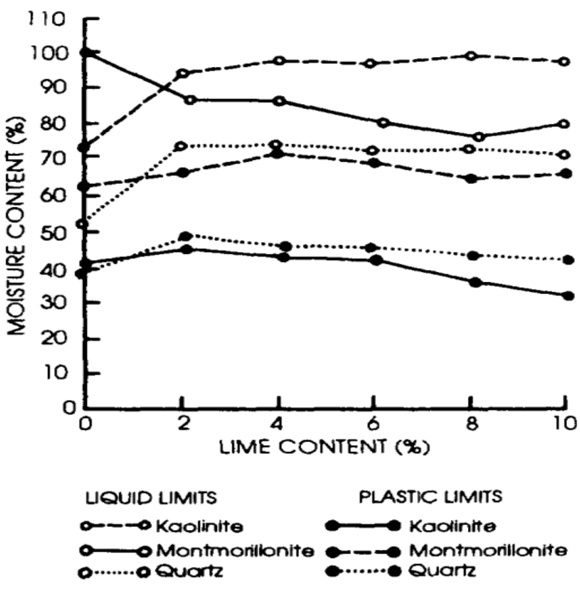


Figure 13: Liquid and Plastic limits of Kaolinite and Montmorillonite (Bell, 1996).

de Brito Galvão, Elsharief, & Simoes (2004), studied on lime stabilization of two tropical residual soils (a brown saprolitic soil as soil 1 and a red lateritic soil as soil 2). Out of the two soils, for soil 1 it was observed that LL and PL increase slightly on lime addition. However, for soil 2 LL decreased while there was no change in Pl observed. For soil 1 no change in PI was observed and for soil 2 a decrease in PI was observed as shown in Figure14. Dash & Hussain (2012), studied an expansive soil (ES) and a non-expansive residual soil (RS). The variations in LL and PL for the two soils are shown in Figure 15 and Figure 16. A decrease in LL was observed for the expansive soil with the lime content, but in the residual soil, LL dramatically increased with the lime content and curing time. Author reports that this behaviour is due to the production of water holding gelatinous products formed during pozzolanic reactions and this behaviour is typical for silica-rich soils. Plastic limit was clearly shown to increase with the lime content and in silica-rich soils this increase is dramatically high.

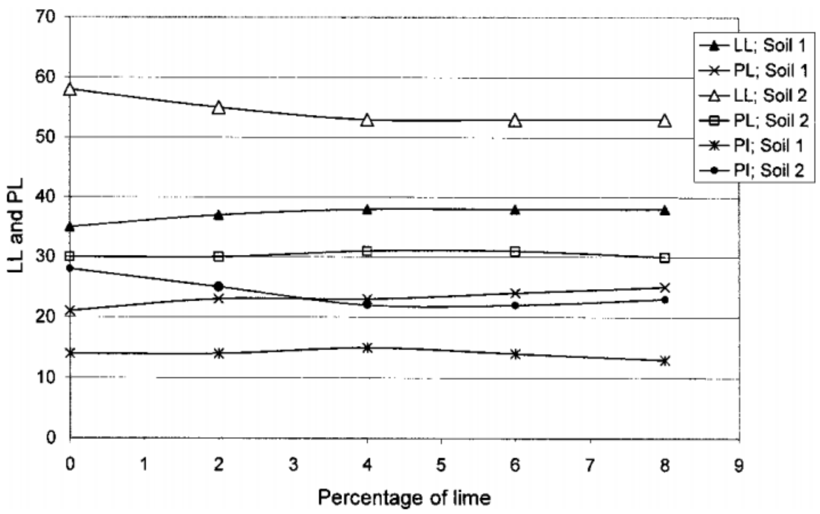


Figure 14: Variation of Liquid limit and Plastic limit with lime content for two tropical residual soils (de Brito Galvao et al., 2004).

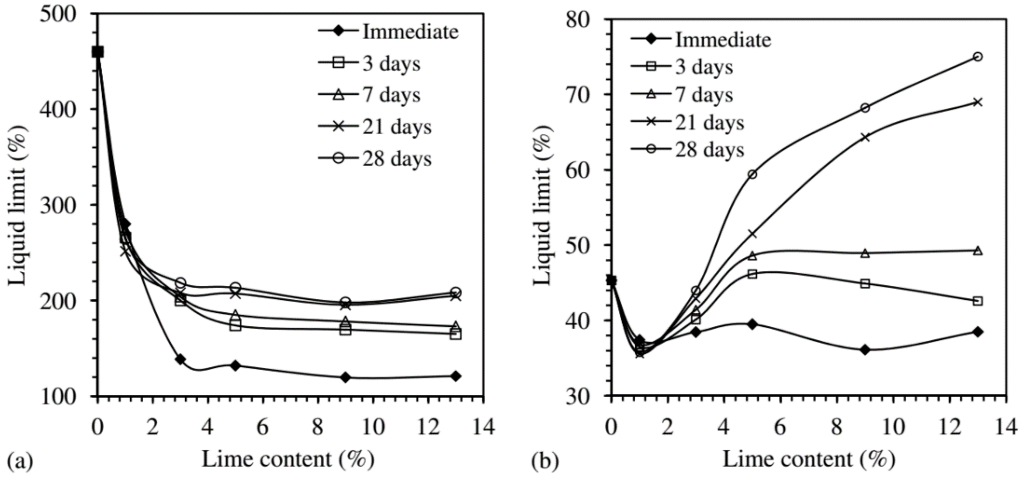


Figure 15: Variation of liquid limit with lime content: (a) expansive soil, (b) residual soil (Dash & Hussain, 2012).

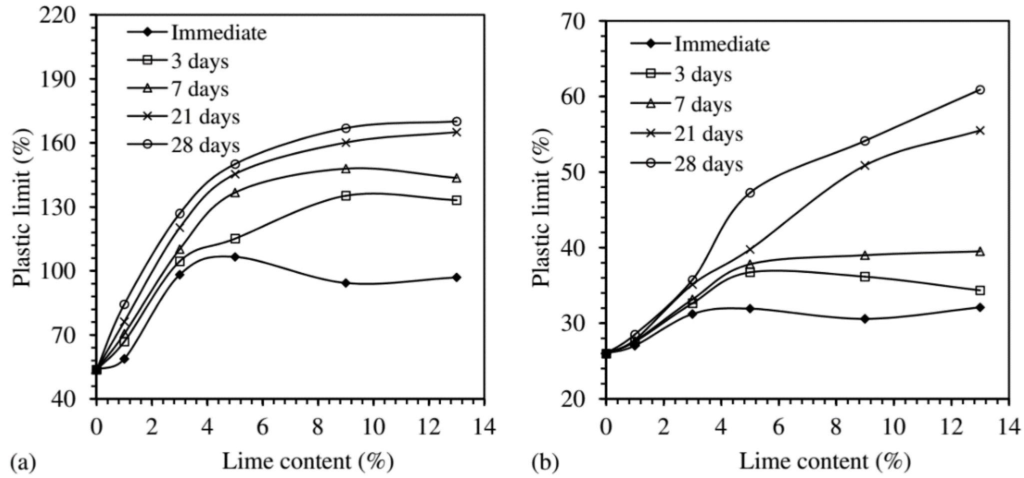


Figure 16: Variation of plastic limit with lime content: (a) expansive soil, (b) residual soil (Dash & Hussain, 2012)

Kumar & Sharma (2004), and Dissanayake et al. (2017) observed a decrease in PI with the addition of fly ash to an expansive soil. This behaviour is due to the cation exchange and flocculation reactions in the soil. In Nawagamuwa & Prasad (2017), an increase in PI with 2% fly ash and then a decrease in PI were observed with the fly ash percentage. Change in LL, PL and PI for an expansive soil treated with different percentages of lime and class F fly ash are shown in Figure 17. A clear decrease in PI was observed when the additive percentage was increased.

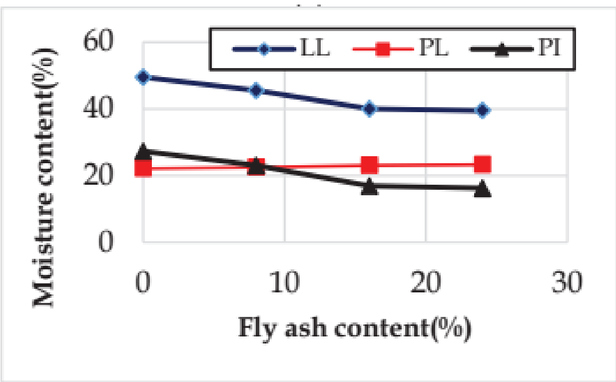


Figure 17: Variation of consistency limits with fly ash (Dissanayake et al., 2017).

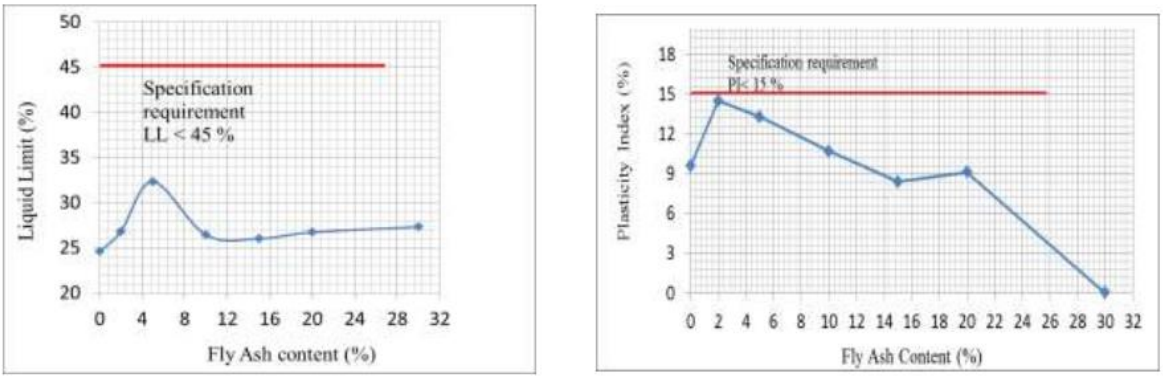


Figure 18: Variation in Liquid limit and Plasticity index with fly ash (Nawagamuwa & Prasad, 2017)

Content (%)		Grain size distribution			Atterberg limits (%)			Specific gravity, G_s	Soil classification	Activity
L	F	Clay (%)	Silt (%)	Sand (%)	L_w	L_p	I_P			
0	0	55.2	42.4	2.4	59.77	27.54	32.23	2.72	CH	0.58
0	3	52.7	44.3	3.0	58.16	29.23	28.93	2.71	CH	0.55
0	6	50.6	46.1	3.3	57.26	31.04	26.22	2.68	MH	0.52
0	9	49.2	47.2	3.6	55.06	32.51	22.55	2.66	MH	0.45
0	12	48.1	48.0	3.9	53.74	33.31	20.43	2.65	MH	0.42
0	15	47.3	48.7	4.0	52.38	35.07	17.31	2.63	MH	0.36
1	3	52.3	44.6	3.1	57.29	30.43	26.86	2.70	MH	0.51
1	6	50.5	46.2	3.3	55.71	32.83	22.88	2.68	MH	0.45
1	9	48.7	47.6	3.5	53.29	33.55	19.74	2.66	MH	0.40
1	12	47.8	48.2	4.0	52.09	35.24	16.85	2.65	MH	0.35
1	15	47.0	48.9	4.1	51.34	36.28	15.06	2.63	MH	0.32
2	3	50.3	46.3	3.4	56.38	31.47	24.91	2.69	MH	0.49
2	6	49.5	47.0	3.5	54.83	32.92	21.91	2.67	MH	0.44
2	9	48.2	48.1	3.7	53.27	34.13	19.14	2.65	MH	0.40
2	12	47.1	48.9	4.0	52.09	34.96	17.13	2.64	MH	0.36
2	15	47.0	48.9	4.1	50.81	36.22	14.59	2.62	MH	0.31

Figure 19: Effect of lime (L) and fly ash (F) on plasticity characteristics (Zha, Liu, Du, & Cui, 2008).

2.3.2. Compaction characteristics

A clear decrease in MDD and an increase in OMC were observed with the addition of lime in many literatures (Ladd, Moh, & Lambe (1960), Bell (1996), Osinubi (1998), Harichane, Ghrici, Kenai, & Grine (2011)). The reason for this behaviour is due to aggregation of particles to occupy space, the low specific gravity of lime and pozzolanic reactions between clay and lime (Harichane et al., 2011). Even though there are no compaction optima from lime stabilization, according to Bell (1996), this should not be considered as a disadvantage as the strength increase with the lime content can compensate for the decrease in MDD. Figure 20 shows the change in optimum moisture content and maximum dry density observed by Bell (1996).

Material	Optimum lime content (%)	Optimum moisture content (%)	Maximum dry density (Mg/m^3)	CBR (%)
Kaolinite	0	29	1.4	1
	6	31	1.33	14
Montmorillonite	0	20	1.29	9
	4	25	1.15	18
Quartz	0	28	1.41	1
	6	32	1.40	22

Figure 20: Comparison of compaction and California bearing ratio test results (Bell, 1996).

Osinubi (1998) studied the effect of compaction effort and compaction delays in lime modified laterite soils. Change in maximum dry density of the soil treated with lime for two different standard energies (Standard Proctor effort and West African effort) are shown in Figure 21. It was observed that the MDD decrease. The effect of compaction delays is shown in Figure 22. With the compaction delay, a decrease in MDD observed and it is concluded that this behaviour is due to compaction energy is utilized for disruption of aggregated particles due to cementation.

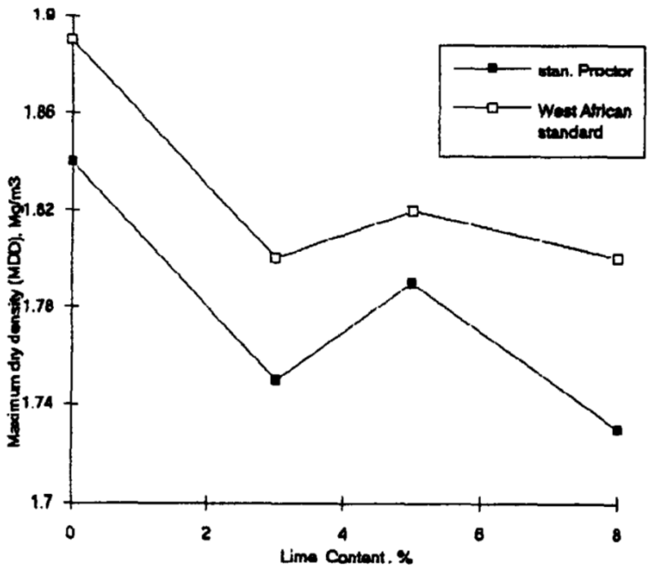


Figure 21: Variation of MDD with lime at no compaction delay (Osinubi, 1998).

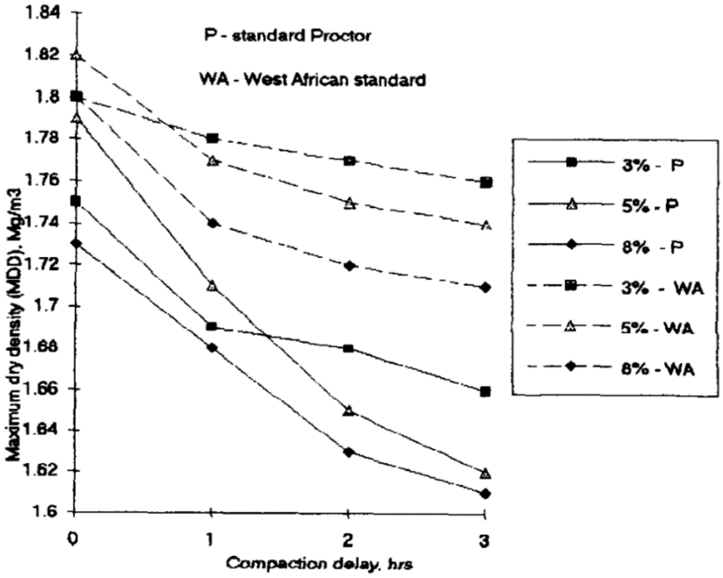


Figure 22: Variation in the Maximum dry density of lime stabilized soils with delayed compaction (Osinubi, 1998).

(Nawagamuwa & Prasad, 2017) and (Dissanayake et al., 2017) observed an increasing and decreasing trend in MDD and a decreasing and increasing trend in OMC with fly ash. Phanikumar (2009), observed an increase in MDD and a decrease in OMC with fly ash on expansive clay. According to Phanikumar (2009), the reason for this behaviour is flocs can roll over themselves more easily during the process of compaction. In the study of Zha, Liu, Du, & Cui (2008), it was observed that the MDD decrease with lime-fly ash mixed with soil.

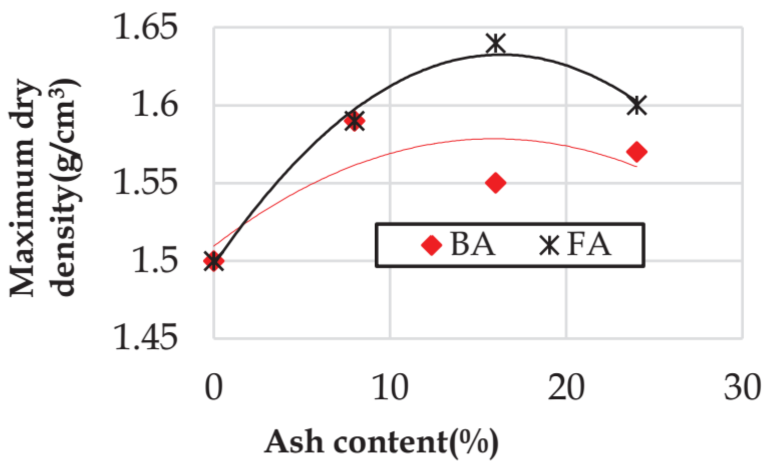


Figure 23: Variation of MDD with fly ash (Dissanayake et al., 2017).

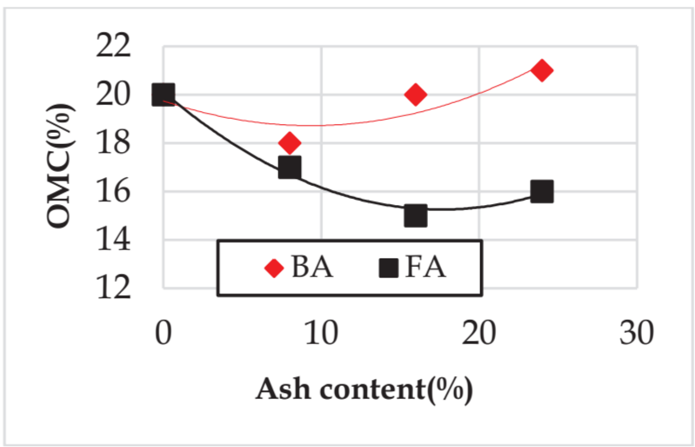


Figure 24: Variation of OMC with fly ash (Dissanayake et al., 2017).

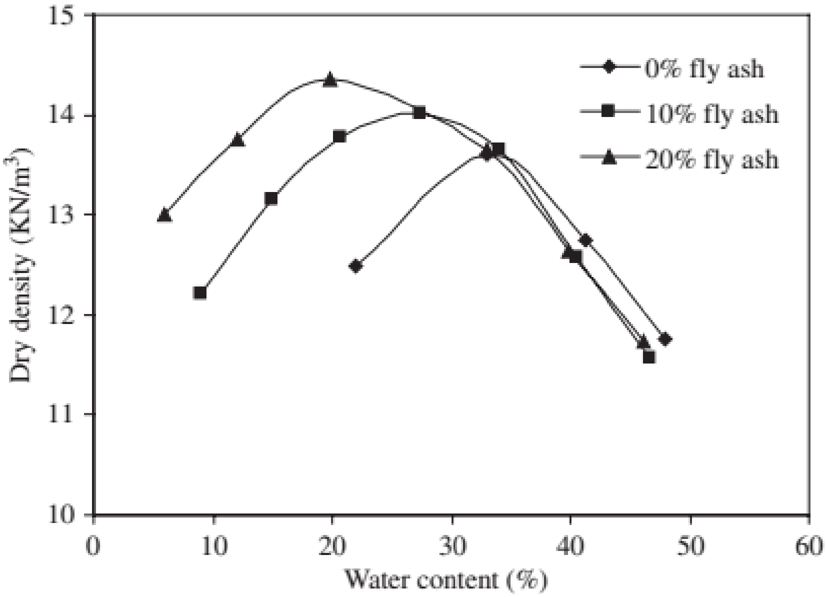


Figure 25: Variation of MDD with fly ash percentage (Phanikumar, 2009).

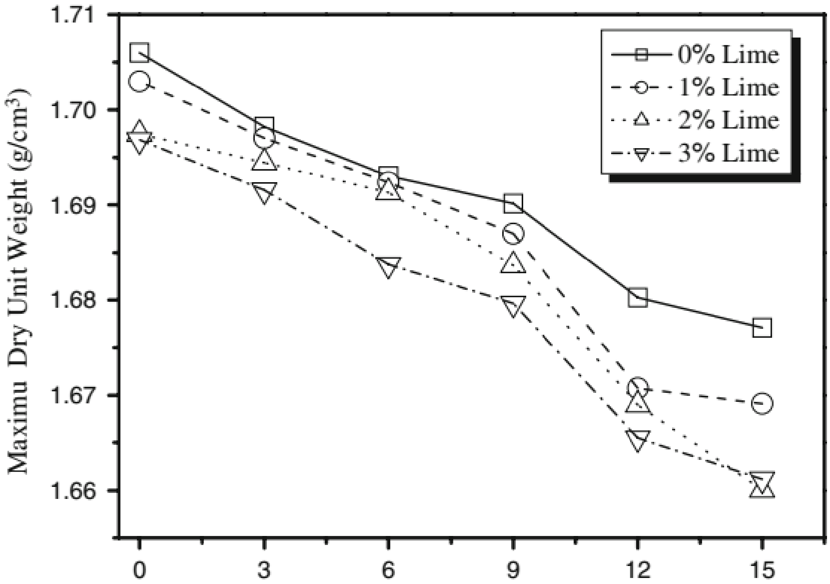
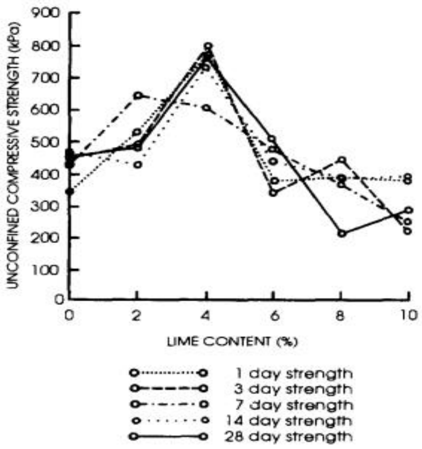


Figure 26: Variation of MDD with lime-fly ash admixtures (Zha et al., 2008).

2.3.3. Unconfined compressive strength

UCS is one of the tests that can be used to obtain the optimum additive percentage for soil stabilization. According to Mitchell & Hooper (1961), the strength of soil-lime mixtures depends on variables such as soil type, lime content, lime type, curing time and method, water content, unit weight and time between mixing and compaction. UCS has been carried out by many researchers (Bell (1996), Dash & Hussain (2012), Ansary, Noor, & Islam (2007), Dissanayake et al. (2017), Elkady (2016), Kaniraj & Gayathri (2003)) and here only salient features of few of the papers will be reviewed.

The increase in strength is due to the formation of cementitious compounds from pozzolanic reactions. It has been observed that with lime UCS would increase to a maximum and further addition of lime will result in a decrease in the UCS (Bell (1996), Dash & Hussain (2012)) as shown in Figure 27 and Figure 28. Furthermore, with the curing time, UCS increases. According to Bell (1996), the decrease in UCS after excessive addition of lime is due to inadequate friction and cohesion in lime, it serves as a lubricant. Dash & Hussain (2012), attributed this behaviour is due to the excess formation of high porosity silica gel which reduces the strength gain through cementation. Osinubi (1998) investigated the effect of compaction delay up to 3h, on UCS in lime treated soils. It was observed that with the compaction delay UCS of lime treated soils declined as shown in Figure 29.



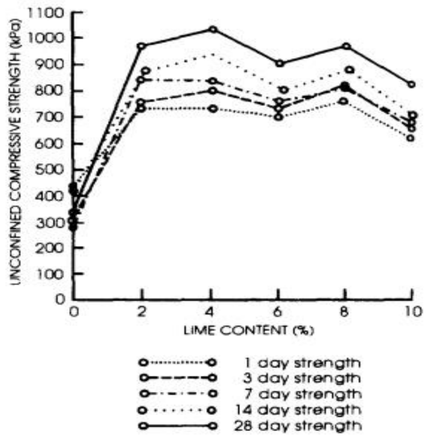


Figure 27: Unconfined compressive strength: a) Montmorillonite b) Kaolinite (Bell, 1996).

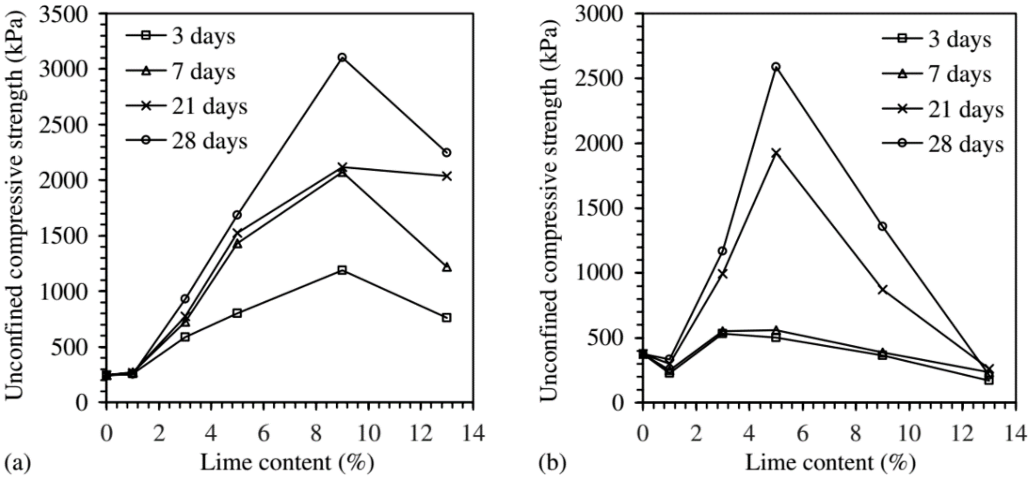


Figure 28: Unconfined compressive strength of lime stabilized: a) expansive soil b) residual soil (Dash & Hussain, 2012).

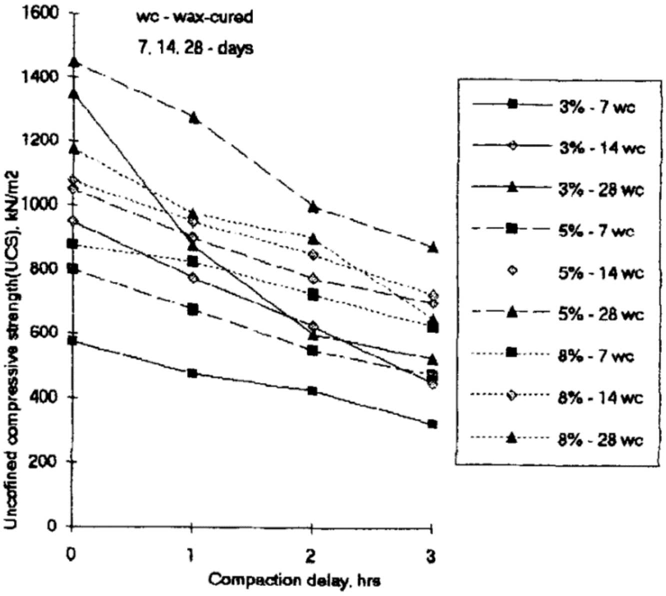


Figure 29: Effect of compaction delay on UCS of lime treated soil (Osinubi, 1998).

Dissanayake et al. (2017) studied the effect of fly ash stabilized on UCS. It was observed that up to 16% fly ash UCS increased with the fly ash percentage and then it declined, as shown in Figure 30. However, UCS increased with the curing time.

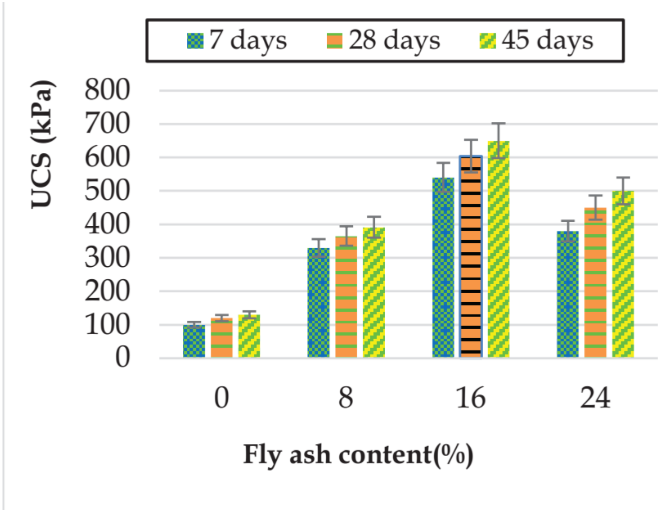


Figure 30: Variation of UCS for fly ash treated expansive soil (Dissanayake et al., 2017).

UCS of lime-fly ash treated soils with small amounts of lime was studied in Zha et al. (2008). With fly ash percentage of 9- 12% the 7 days cured UCS of lime-fly ash treated soil increased. Beyond that, the UCS decreased. This attributed that fly ash up to optimum content can increase the strength due to pozzolanic reactions and cemented materials, but further addition would decrease UCS as fly ash acts as unbound silt particles which have neither appreciable friction nor cohesion (Zha et al., 2008).

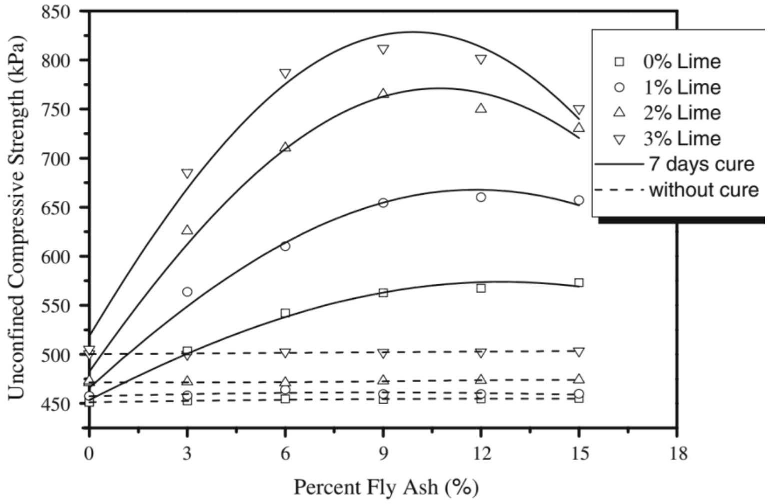


Figure 31: Variation of UCS of lime-fly ash admixture treated expansive soil (Zha et al., 2008).

2.3.4. California bearing ratio (CBR)

According to Indraranta (1994), CBR values closely related to both compressive strength and bearing capacity of compacted subgrade. Bell (1996), observed an increase in CBR value for different clay minerals treated with lime as shown in Figure 20. In the study of Osinubi (1998), on CBR value of a laterite soil compacted with two different energies and with different compaction delays, it was observed that the CBR value was decreased with the delay in compaction. The reason for such a decline was attributed to cementitious compounds formed before compaction has to be disrupted.

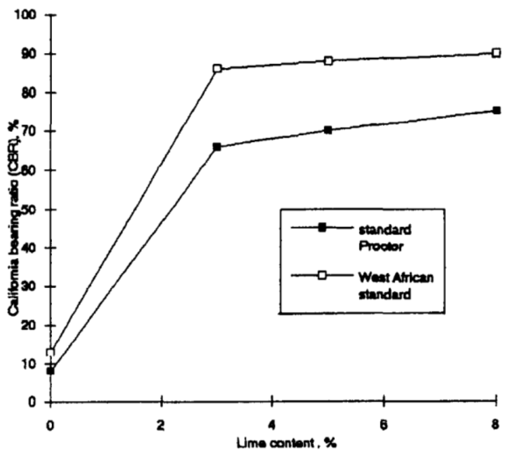


Figure 32: Variation of CBR with lime at no compaction delay (Osinubi, 1998).

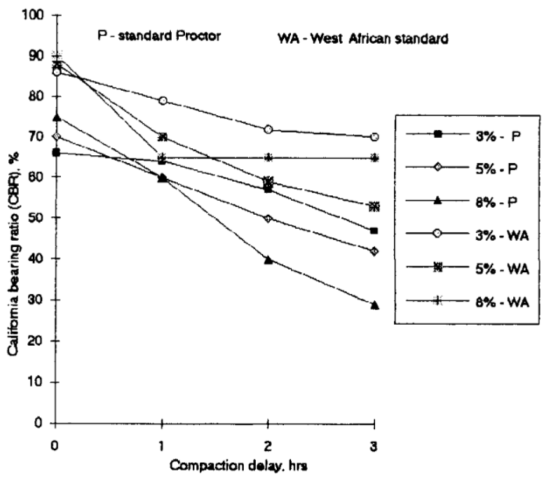


Figure 33: Variation of CBR of lime treated lateritic soil with compaction delays (Osinubi, 1998)

In the study of Nawagamuwa & Prasad (2017), it was observed that soils having low CBR can be stabilized with fly ash to improve the CBR value, but adding fly ash beyond the optimum fly ash percentage would result in a decrease in CBR value.

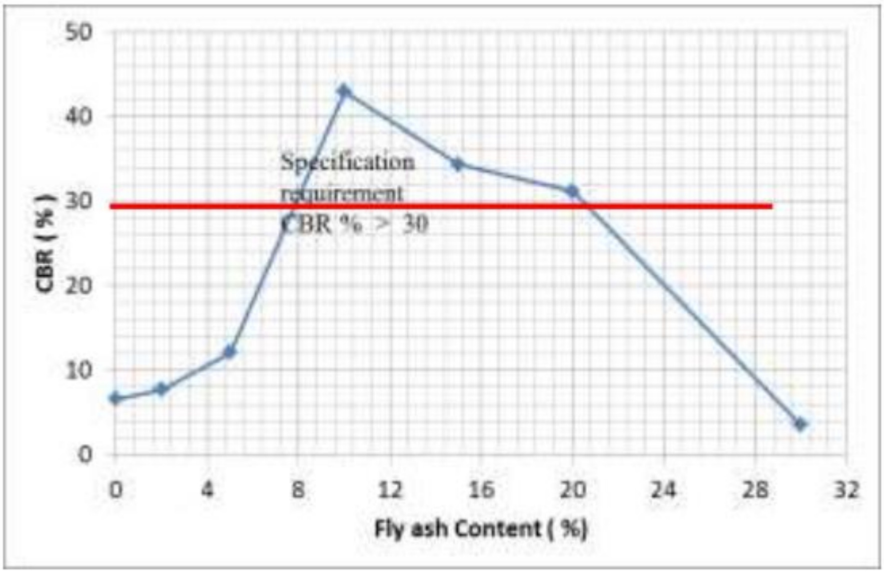


Figure 34: CBR with different fly ash contents (Nawagamuwa & Prasad, 2017).

Prasad, Sahoo, & Kumar (2010), studied the effect of different fly ash and lime-fly ash proportions for the improvement in CBR value. It was observed, fly ash added in excess of 15% resulted in a decline in CBR and for 15% fly ash, if the lime content was increased beyond 4%, even then the CBR value decreased.

2.4.Summary of Previous Literature

Table 4: Summary of previous literature

1. Lime stabilization of clay minerals and soils			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Bell, 1996)	Stabilization of clay minerals namely kaolinite, montmorillonite and quartz.	The soil was stabilized using 2, 4, 6, 8, 10% lime. The properties of improved clay were identified using Proctor compaction test, consistency limits, CBR, SEM and UCS	Lime increased the plasticity of kaolinite and quartz but decreased in montmorillonite. There was an increase in optimum moisture content and a decrease in dry density. The CBR upon addition of lime increased. Some notable increase in Young's modulus was observed when treated with lime. Curing time period and temperature of curing had an influence on strength developed.
2. The processes involved in the lime stabilization of clay soils			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Croft, 1964b)	Crystalline reactions products formed at 40^0 C temperatures were studied and the mechanisms of the reactions described.	Using X-ray diffraction and chemical analysis of different clay minerals. The physical changes were also observed with time.	Higher temperatures have been responsible for good strength in both long term and short term and long term. Compaction should be immediate. Sooner the compaction betters the final strength. Prolonged delays will certainly be detrimental. The ideal amount of water to add is not OMC but more than that. However, adding more water will defeat the purpose of compaction.

			Therefore, the optimum amount should be used. For short time periods, the LL increase or remains the same for Kaolinites. The rise in LL is due to the action of OH^- ions modifying the affinity of the clay surface with water. In clays, PL was observed to be rising with ageing. In expansive soils the rate of formation of reaction products is high.
3. A critical appraisal of the role of clay mineralogy			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Cherian & Arnepalli, 2015)	Conventional methods in finding the OLC and the chemistry behind lime stabilization were studied.	Based on the theoretical and experimental observations of conventional tests.	It is gathered that the prevailing OLC determining rules and theories are less conservative.
4. Influence of compactive efforts and compaction delays on Lime-treated soil			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Osinubi, 1998)	Effect of compaction effort and the delay in compaction up to 3h on the compaction strength of lateritic soils treated with a maximum of 8% lime.	Compaction effort was changed using the methods specified in proctor compaction where a low effort is used and using the West African standards for intermediate effort. Then the effect of compaction delay	MDD decrease with the delay in compaction as well as the optimum moisture content. UCS and CBR decreased in value with an increase in compaction delays for both compaction efforts.

		with the change of delay in compaction was observed.	
5. A quick test to determine lime requirements for lime stabilization			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Eades & Grim, 1966)	In this study, the optimum lime requirement for soil stabilization is studied and a quick test to determine the lime requirement is given.	Measuring the lime content to achieve a pH of 12.4. This was tested for a long period of time. Compressive strength change with curing time and pH was also studied.	Measuring pH after 1h of soil lime mixture will help in finding the lime required for stabilization. After 1h the lowest lime percentage to make pH 12.4 is the minimum amount of lime required to satisfy the lime requirement is maintained.
6. Utilization of Lime for Stabilizing soft Clay Soil of High Organic Content			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Sakr, Shahin, & Metwally, 2009)	Geotechnical and mineralogical investigation on improving a clay soil with 14% organic content using lime	Laboratory experiments were conducted adding 1,3,5,7 % lime by weight and curing for 7,15,30,60 days. In this particle size analysis, plasticity limits, unconfined compressive test and oedometer test were conducted	Lime results in formation of new cementing materials. A gradual increase in particle size with lime % and the curing time as the formation of lumps has observed. LL increases with lime percentage and decreases with the curing time. Unconfined compressive strength improves upon addition of lime and with the curing time. This study proves that the soft clay with high organic content can be stabilized with 7% lime satisfactorily.

7. Effect of adding natural pozzolana on geotechnical properties of lime-stabilized clayey soil			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(al-Swaidani, Hammoud, & Meziab, 2016)	Effect of adding natural pozzolana to lime stabilized clayey soils (CH type).	Consistency limits, compaction, California bearing ratio (CBR) and linear shrinkage properties were investigated. Natural pozzolana and lime are added to soil within the range of 0%-20% and 0%-8%, respectively	It was concluded that natural pozzolanas and lime can be effectively used for road stabilization. Using natural pozzolana reduce the cost and reduce the CO_2 emission when compared with other products. Results from SEM and EDX shows that there is a significant change in microstructure of treated clayey soils
8. Comparison of the Stabilization Behaviour of Fly Ash and Bottom Ash Treated Expansive Soil			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Dissanayake et al., 2017)	The effect of class F fly ash and bottom ash stabilization on an expansive soil was studied.	Laboratory experiments with 8, 16 and 24% fly ash were conducted. The change in compaction characteristics, Atterberg limits, UCS, swell pressure and microstructure was studied using SEM.	LL and PI decreased while PL was increased. OMC decreased up to a minimum of 16% fly ash addition and 8% bottom ash addition and then increased. MDD increased up to 16% fly ash addition and then decreased. UCS increased up to 16% fly ash addition and then decreased with ash content. Both fly ash and bottom ash can be used for expansive soil stabilization but fly ash is better.

9. Effect of Fly Ash on Engineering Properties of Expansive Soils			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Kumar & Sharma, 2004)	The efficiency of class F fly ash as an additive for stabilization of an expansive soil.	Effect of fly ash on swelling characteristics, plasticity, compaction, strength and hydraulic conductivity. Fly ash was added as 0, 5, 10, 15 and 20% of dry weight basis.	The addition of fly ash reduces the plasticity index while LL decreases. With the increase in fly ash content OMC decreases and MDD increases

10. The Pozzolanic Reactivities of some New South Wales fly ashes and their application to soil stabilization			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Croft, 1964a)	The nature of reaction products formed in lime-fly ash mixtures.	Reactivity of number of fly ash lime mixtures from different power plants and their potential pozzolanic values were studied.	Preliminary examinations showed that the lime-fly ash mixtures were slow to react. Effect of lime and fly ash addition raised pH above 12 and hence increase the solubility of SiO_2 and Al_2O_3. If the clay is not active or the fines tend to a non-plastic condition short-term strength will depend largely on particle grading.

11. Effect of fly ash stabilization on strength properties of contaminated clay sand soils			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Saeed, 2016)	In this study, fly ash was used as binder to stabilize lead and chromium present in clayey sand	Fly ash and lime was used as admixtures and keeping the lime content at 5% the fly ash content	It was observed that the strength was enhanced with the fly ash increase and with the curing time. This was attributed to the

	and soils as artificial pollutants	was changed to 5, 10 and 15%.	formation of pozzolanic cementitious CASH (Gismonide) with the time. This compound was responsible on the cementation of the lime, fly ash, heavy metal and soil matrix and then lead to enhancement in the strength of this matrix.
12. Effect of fly ash stabilization on geotechnical properties of Chittagong coastal soil.			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Ansary et al., 2007)	Strength properties of soil stabilized with two fly ash products.	Strength tests were conducted on the specimens of 28day curing. For the stabilization fly ash was mixed with 6, 12 and 18% of fly ash with a fixed lime quantity of 3%.	The results from the experiment show that increasing the amount of fly ash will increase the strength when compared to untreated samples depending on the fly ash content and curing age. Flexural strength, as well as flexural modulus, increased with the amount of fly ash used.
13. The stabilization Sydney basin Wiananatta derived residual clay with fly ash and chemical control of environment.			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Nettleton , 1963)	Compaction and strength characteristics of a clay soil with the addition of fly ash from 0-20%	Clay behaviour with the addition of fly ash for different environmental conditions such as fresh water, sea water and acid water was determined.	Variation pH and electrolyte concentration have no effect on maximum dry density. There is no short-term pozzolanic action (7-14 days) resulting in increased strength with fly ash. Good results for stabilization can be obtained with 15 to 29% fly ash.

			Addition of small quantities of lime will increase pozzolanic activity and result in great strength. Small quantities of lime will be more efficient rather than cement.
14. Strength characteristics of Fly ash mixed with lime stabilized soil			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Prasad et al., 2010)	Effect of fly ash mixed with small amount of lime on the strength characteristics of soil for construction.	A series of laboratory tests Proctor compaction test, triaxial test, CBR and UCS were conducted on soil specimens treated with different percentages of fly ash and fly ash-lime mixtures.	With the increase in fly ash and lime content OMC increases and MDD decrease. There is a marginal increase in cohesion and friction angle of soil with the increase of fly ash and lime up to 15% and 4% respectively. The maximum increase in UCS was obtained with 15% fly ash and 4% lime. Beyond that, the UCS decreased. Similar behaviour was observed in CBR
15. Evaluation of lime and fly ash stabilization of soils by compressive strength tests			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Moh et al., 1955)	Develop a method to evaluate lime and fly ash stabilization Merits of lime-fly ash stabilization	Experiments were made to identify the different ways of preparing, curing and testing of lime-fly ash stabilized soils. Silty and clayey soils were used.	Maximum dry density usually decreases and optimum moisture content was increased. With the increase in the amount of lime and fly ash keeping the ratio of lime: fly ash constant showed that the compressive strength increases. The stability of lime-fly ash

			mixtures increased with the ageing in the presence of moisture due to the pozzolanic action.
16. Stabilization of fine and coarse grained soils with lime and fly ash admixtures			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Goecker et al., 1956)	Study on the unconfined compressive strength, proctor moisture density, the consistency limits, pH and the resistance to freezing and thawing.	Experiments were made to identify the different ways of preparing, curing and testing of lime-fly ash stabilized soils. Silty and clayey soils were used.	Maximum dry density usually decreases and optimum moisture content was increased. Strength increased with the ageing. Elevated temperature curing increases the strength. There is no relationship between strength and relative humidity. Improved the consistency limits upon addition of lime and fly ash.
17. The effect of curing conditions on the unconfined compression strength of lime-treated expansive soils			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Elkady, 2016)	In this study, the effect of different curing environments on the UCS of lime treated soils was studied.	Under different curing environments, remoulded samples with 2, 4 and 6% of lime were cured for 7, 14 and 28 days.	For all the environments there was an increase in UCS with curing period. The increase in normally cured samples is due to cementation, but in other environments, the suction stresses also contribute to the strength.
18. Comparison of the effect of mixing methods (dry vs.wet) on mechanical			

and hydraulic properties of treated soil with cement or lime			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Pakbaz & Farzi, 2015)	Comparison of cement and lime-treated soil using wet and dry method treatment	Using dry and wet methods a saturated sand mixture treated with 2,4,6,8 and 10% cement, lime The treated soil sample were cured for 7, 14 and 28 and UCS and consolidation were tested.	The UCS of wet cement treated samples were higher than dry treated samples and for lime dry treated samples got higher strength. Lime treatment resulted in a higher elastic modulus than cement treatment and dry treatment caused a higher elastic modulus

19. Improving soils of low CBR with fly ash for road applications			
Author	**Factors studied**	**Method adopted**	**Conclusion**
(Nawagamuwa & Prasad, 2017)	Use of fly ash in the improvement of low CBR soils for road constructions.	Laboratory tests such as particle size distribution, Atterburg limits, standard Proctor compaction and CBR tests were conducted on soil samples with different fly ash percentages (2, 5, 10, 15, 20 & 30%).	It was observed that with 10% fly ash the required specifications in ICTAD were achieved.

2.5.Research Gap

The literature review covered lime and fly ash stabilization of soils. Previous researches have conducted research on the stabilization of soils using different additives such as lime, cement, fly ash, natural pozzolans, cement kiln dust, fibres, rice husk, polymers etc. Laboratory experiments have been conducted to investigate the improvement of different soil characteristics (gradation, plasticity, compaction, California bearing ratio (CBR), permeability, compressibility, unconfined compressive strength (UCS), triaxial tests, etc.) with different percentages of additive or additives. To investigate the mineralogical changes and evolution of the reactions based on new product formed microstructural tests have been conducted using scanning electron microscope (SEM), energy dispersive spectrometry (EDS), X-ray diffraction (XRD) etc. The properties that should be investigated were decided according to the use for the improved soil.

Many tests were conducted, to identify the additive percentages for the use in different areas such as road subgrade, base construction, to use embankment fill materials, as landfill covers etc. The soils such as peats, organic clays, highly expansive soils, silts, residual soils were used for these experiments. To the knowledge of the author, limited studies have been conducted on the stabilization of unsuitable residual clayey soils of low expansive nature using lime and fly ash. Therefore, in the current study, experiments were carried out with different mix proportions of lime and fly ash to stabilize low expansive residual clay.

3. EXPERIMENTAL STUDY

3.1.Objectives

To investigate the effect of different lime, fly ash and lime-fly ash admixture percentages, on unsuitable residual soil properties such as plasticity, compaction, unconfined compressive strength (UCS) and California bearing ratio (CBR), for road pavements.

3.2.Methodology

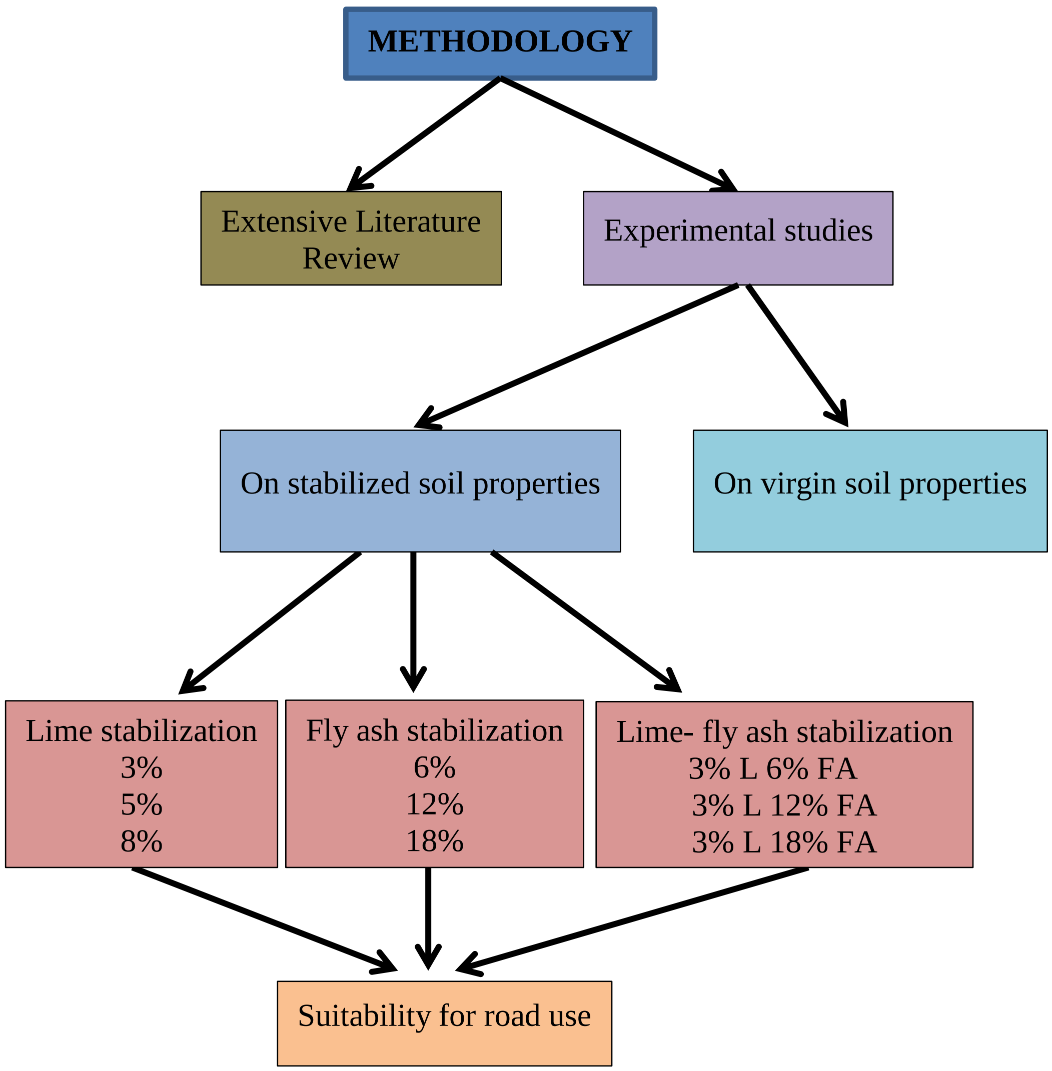


Figure 35: Summary of the methodology

- Investigate the basic soil properties of the unsuitable soil such as particle size distribution, plasticity characteristic, compaction characteristics and specific gravity.
- Investigate the properties of lime and fly ash
- Study the variations of the residual soil with different mix proportions of lime and fly ash.
- Investigate the suitability of stabilized soil to be used in road pavements.

3.3.Materials

3.3.1. Characteristics of the soil

An unsuitable residual clayey soil, which was left unused in Mirigama area from Central Expressway project, Sri Lanka, was obtained for the stabilization. First, any vegetation present in the soil was removed and the sample was air dried. Then the soil was mixed well and soil lumps were broken without crushing the particles using wooden mallets and rubber pestles. Before testing, soil samples were sieved through appropriate sieves and kept in the oven for 24 h to get moisture free. First, the soil was tested to identify the characteristics of the virgin soil, such as particle size distribution, Atterburg limits, maximum dry density, optimum moisture content, and specific gravity.

Figure 36: Breaking soil lumps using a rubber pestle and a wooden mallet.

The particle size distribution for the soil was obtained from sieve analysis and hydrometer analysis (ASTM D422, 2004). 500 g of oven dried soil sample was used for the sieve analysis and a 50 g oven-dried sample sieved through 0.425 mm sieve was used for the hydrometer analysis. The liquid limit and plastic limit of the soil was determined according to (ASTM D4318, 2004). For the liquid limit determination multipoint method (Method A) was used, for a range of moisture contents. Furthermore, using the standard Proctor compaction test (ASTM D698, 2004) the maximum dry density and the optimum moisture content of the soil was obtained. Out of three alternative methods specified Method A was used where the sample was sieved through 4.75 mm sieve and compacted in 3 layers by giving 25 blows per each layer in a 101.6 mm (4 in.) diameter proctor mould. The specific gravity of the soil was investigated using (BS1377-4, 1990). The soil was classified according to the unified soil classification system (USCS) (ASTM D2487, 2004).

3.3.2. Lime

Lime used for the experiments was a fine ground calcium hydroxide ($Ca(OH)_2$), produced by a local company.

Figure 37: Lime used for the experiments

3.3.3. Fly ash

Fly ash was obtained from Norochchole, Lakvijaya thermal power plant. The chemical composition for fly ash was obtained from “Test Report No – SS 1710206” submitted to National Engineering Research and Development Centre of Sri Lanka by Materials laboratory, Industrial Technology Institute on 2017-08-31 which was done for the same batch of fly ash used for the experiments. The classification of fly ash was done according to (ASTM C618, 2004) considering the composition.

Figure 38: Fly ash used for the experiments

3.4.Experimental Procedure

The residual clayey soil used for the experiment was stabilized with different mix proportions of lime and fly ash. Different lime and fly ash percentages were added as a percentage of the dry soil mass. ((Dry mass of additives / Dry soil mass) x100 = Additive percentage). The different mix proportions used for the stabilization are shown in Table 5.

Table 5: Different additive percentages used

<table>
<tr><th>Lime percentage</th><th>Fly ash percentage</th></tr>
<tr><td>0</td><td>0</td></tr>
<tr><td>3</td><td rowspan="3">0</td></tr>
<tr><td>5</td></tr>
<tr><td>8</td></tr>
<tr><td rowspan="3">0</td><td>6</td></tr>
<tr><td>12</td></tr>
<tr><td>18</td></tr>
<tr><td rowspan="3">3</td><td>6</td></tr>
<tr><td>12</td></tr>
<tr><td>18</td></tr>
</table>

First, the soil was stabilized for 3, 5 and 8% of lime without adding fly ash. Then the soil was stabilized with 6, 12 and 18% fly ash by dry soil weight. According to (Hilt & Davidson, 1960a) the initial consumption of lime (ICL) can be calculated using Equation 8. From the Equation 8, the initial consumption of lime was calculated as 1.85%. According to (Bell, 1996) 1-3% lime is required to satisfy the cation exchange reactions and any further addition of lime will take part in pozzolanic reactions which will result in strength development. Therefore, the soil was stabilized with 3% lime increasing the fly ash percentage. The improvement in the liquid limit, plastic limit, plasticity index, maximum dry density, optimum moisture content, standard Proctor compaction test, unconfined compressive strength and California bearing ratio was studied for different lime and fly ash percentages following the standards given in Table 6.

Table 6: Standards used for the experiments.

Test	Standards Used
LL and PL	ASTM D4318
Proctor compaction	ASTM D698
UCS	ASTM D2166
CBR	BS 1377-4:1990

Atterburg limits and Proctor Compaction tests were carried out, for different additive percentages, similar to the procedures explained in section 3.1.1. Unconfined compressive strength (UCS) test was carried out to study the strength characteristics. For UCS soil samples were compacted to optimum moisture content and maximum dry density in Proctor mould. Then cylindrical specimens were obtained using sampling tubes of diameter 38 mm. Using the sample extruder and a split mould, cylindrical samples of 38 mm in diameter and 85 mm in height was obtained. After preparation, the specimens were sealed using plastic bags as shown in Figure 39 to prevent carbonation from happening and cured under a controlled temperature of 27±

2 ^{0}C and relative humidity was maintained around 100%. Samples were cured for 7, 14 and 28 days. Samples were tested with a constant strain rate of 1.2 mm/min.

4 day soaked CBR value was obtained compacting the soil and additive mix to optimum moisture content and maximum dry density. The soil was compacted immediately after mixing. CBR samples were tested at a constant penetration rate of 1.27 mm/min.

Figure 39: Prepared sample and curing in plastic bags

4. RESULTS AND DISCUSSION

4.1.Characteristics of soil

The result for the particle size distribution of the residual soil is shown in Figure 40. From the grading curve, it was observed that gravel, sand, silt and clay percentages are 0.8, 38.0, 40.0 and 21.2% respectively. A summary of the characteristics of the residual soil tested is shown in Table 7.

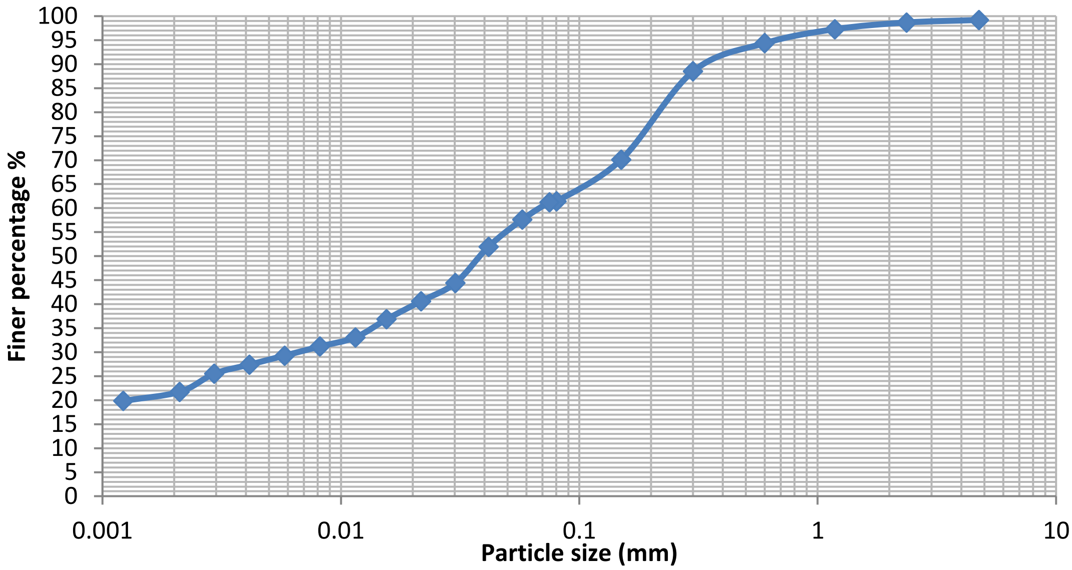


Figure 40: Particle size distribution of the residual soil.

Table 7: Characteristics of the residual soil.

Gravel		0.8%
Sand		38.0%
Fines	Silt	40.0%
	Clay	21.2%
Liquid Limit (LL)		30%
Plastic Limit (PL)		17%
Plasticity Index (PI)		13%
Maximum Dry Density (MDD)		1685 kN/m^3
Optimum Moisture Content (OMC)		17.9%
California Bearing Ratio (CBR)		3.5%
Unconfined Compressive Strength (UCS)		56.8 kN/m^2
Specific Gravity (G_s)		2.60

The fines percentage is greater than 50%, PI is greater than 7 and plots above the A line in plasticity chart as shown in Figure 41. Therefore, according to the Unified Soil Classification System, the soil can be classified as Lean clay (CL).

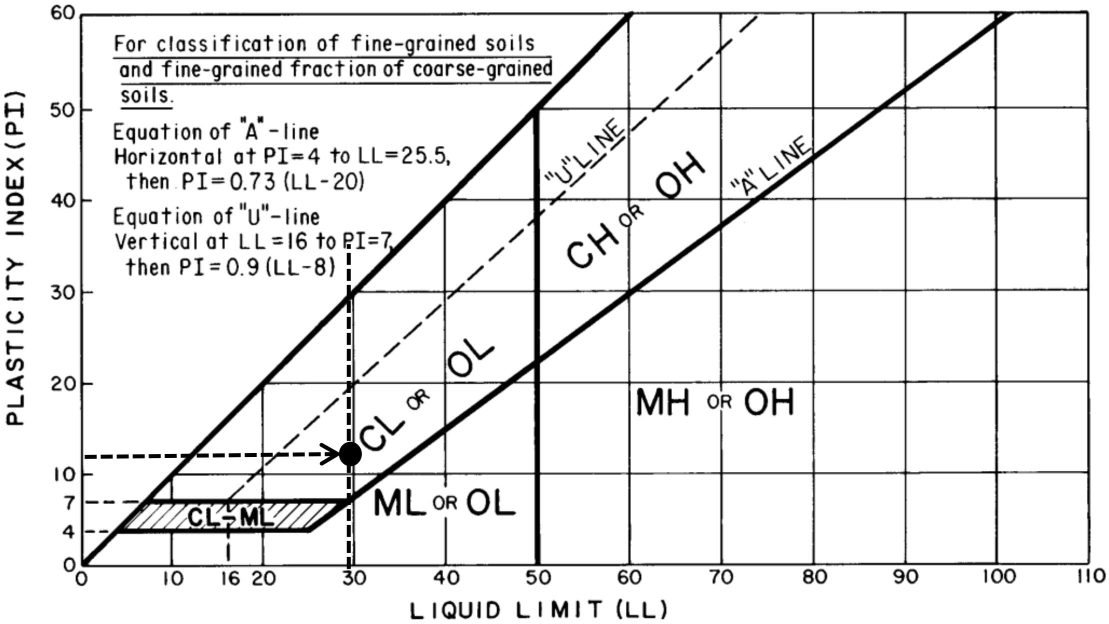


Figure 41: Plasticity chart (ASTM D2487, 2004)

4.2.Lime

A commercially available lime was used for stabilization, of specific gravity 2.54. The particle size distribution analysis result, using laser particle analyser (HMK-CD2 laser particle analyser), for the lime is shown in Figure 43.

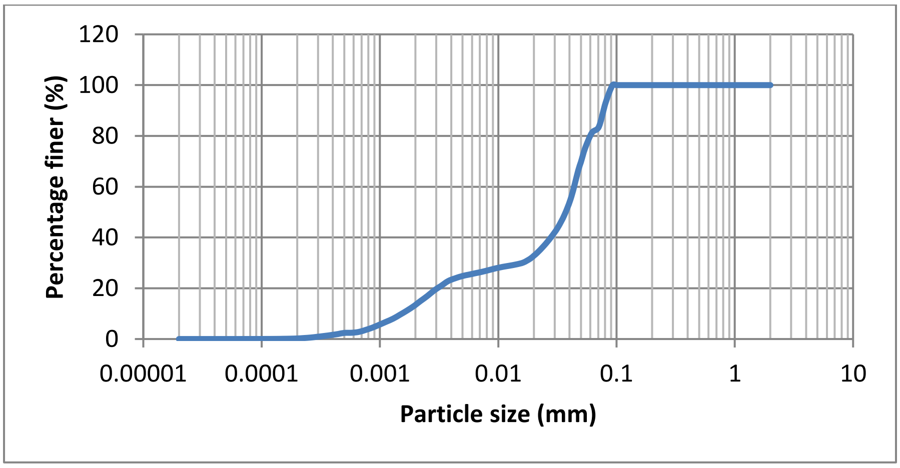


Figure 42: Particle size distribution for lime

4.3. Fly ash

In Table 8 the chemical composition of fly ash is shown. Summation of silicon dioxide (SiO_2), aluminium oxide (Al_2O_3) and iron oxide (Fe_2O_3) is 81.2%. Sulphur trioxide (SO_3) and loss on ignition are 0.5% and 3.7% respectively. Therefore, fly ash can be classified as Class F fly ash according to ASTM C618 (2004). The particle size distribution analysis result, using laser particle analyser (HMK-CD2 laser particle analyser), for the fly ash is shown in Figure 43. It was observed that about 86% of particles are in the range of silt and specific gravity is 2.20.

Table 8: Chemical composition of fly ash

Composition	Percentage by weight
SiO_2	45
Al_2O_3	31.8
Fe_2O_3	4.4
CaO	9
MgO	1.1
SO_3	0.5
Cl-	<0.01
Na_2O	0.4
Loss on ignition	3.7

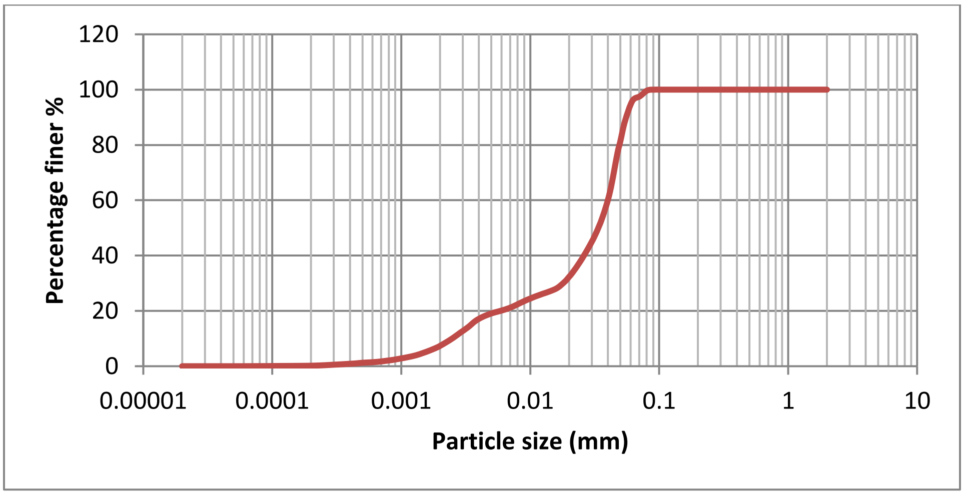


Figure 43: Particle size distribution for fly ash

4.4.Plasticity characteristics

Results for the liquid limit, plastic limit and plasticity index of soil stabilized with 3, 5 and 8% lime by dry soil mass is shown in Figure 44. It was observed that the change in liquid limit was not that significant, but it showed a slightly increasing trend. The plastic limit was observed to be increased from 17.0% to 20.5% with 3% lime addition and then increased slightly with the lime percentage. Plasticity index was decreased by about 3% with 3% lime and then it remained more or less steady. When lime is added to the soil Ca^{2+} cations replace the monovalent cations in diffused double layer (DDL) of negatively charged minerals resulting in a reduction in thickness of the DDL, flocculation and agglomeration. This results in the reduction of plasticity and increase in workability of the soil (Bell, 1996). The lime fixation point or initial consumption of lime for the residual soil used can be identified as 3%, as there is no change in PI with further addition of lime.

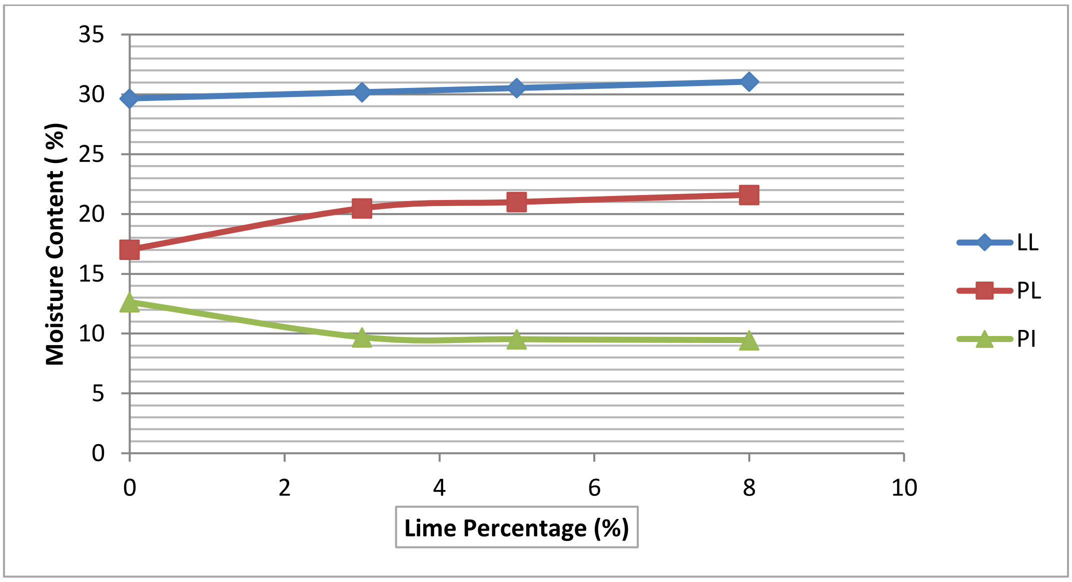


Figure 44: Change in LL, PL and PI for lime stabilization.

As shown in Figure 45 change in LL, PL and PI are not that significant, for fly ash stabilization. It was observed that the PI was almost the same, as the decline was only about 0.3%. Available free Ca^{2+} ions for the cation exchange are very low in fly ash. Therefore, the variation in LL, PL and PI of fly ash treated soils are not significant.

The results for the change in LL, PL and PI with 3% constant lime content, increasing the fly ash percentage by dry soil mass is shown in Figure 46. For 6% fly ash there was a clear increase in both LL and PL, which resulted in no variation in the PI. With further addition of fly ash, there was no significant change in LL, PL or in PI. According to Dash & Hussain (2012), the increase in LL is a result of the formation of water holding gelatinous products typically in soils rich in silica.

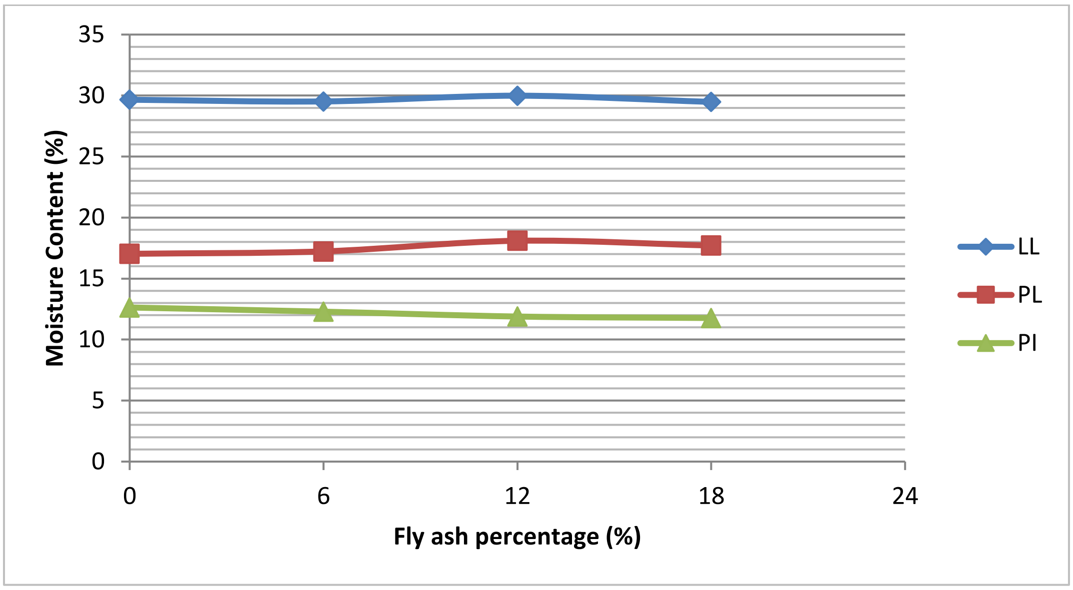


Figure 45: Change in LL, PL and PI for fly ash stabilization

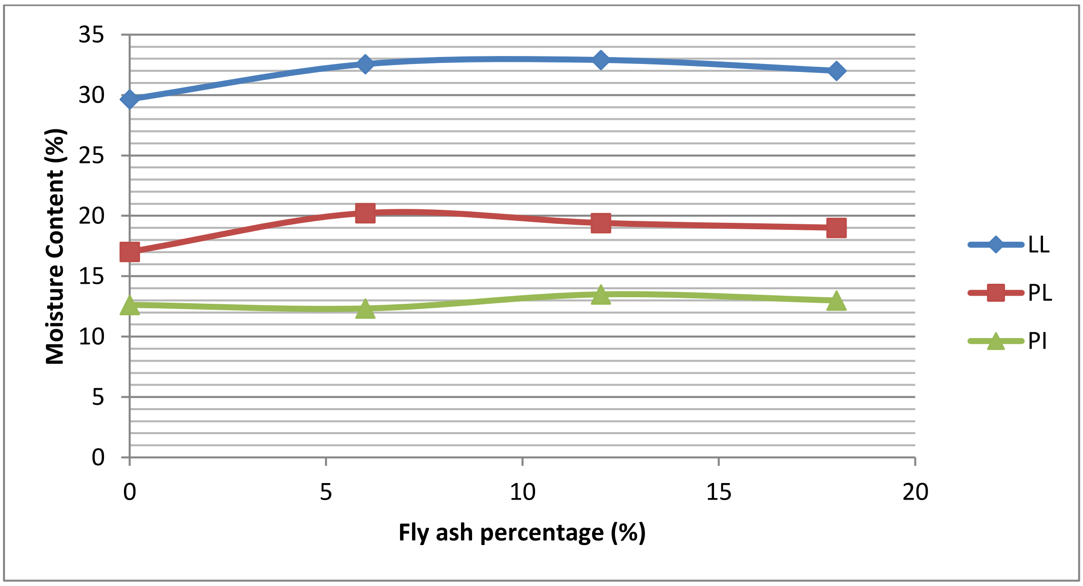


Figure 46: Change in LL, PL and PI for 3% lime and different fly ash percentages.

4.5.Compaction characteristics

For lime stabilization, the change in maximum dry density (MDD) and optimum moisture content (OMC) with the increase in lime percentage are shown in Figure 47 and Figure 47 respectively. It was observed that with 3% lime change in maximum dry density was not that significant, but with further addition of lime, the MDD decreased. With 8% lime, the MDD decreased by about 2.2%. According to Lees, Abdelkader, & Hamdani (1982), the reduction in dry density with lime percentage is due to the formation of cementitious products gradation changes increasing void ratio which reduces compressibility. The low specific gravity of lime is also a cause for the decline in MDD. The change in OMC can be considered as insignificant although there was an increase of about 1% with 5% lime addition.

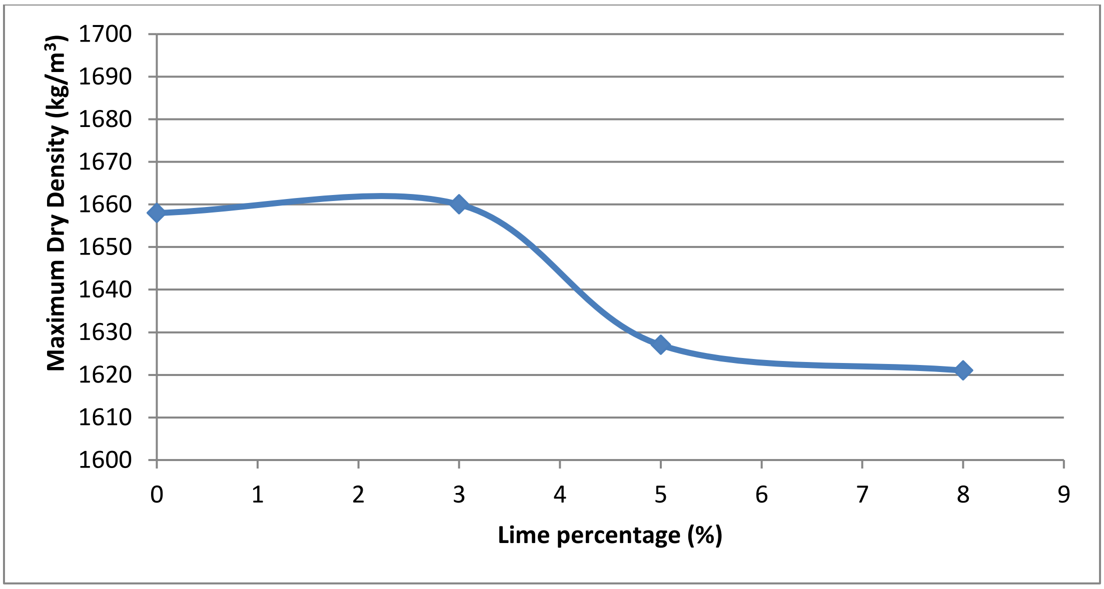


Figure 47: Change in MDD with lime percentage.

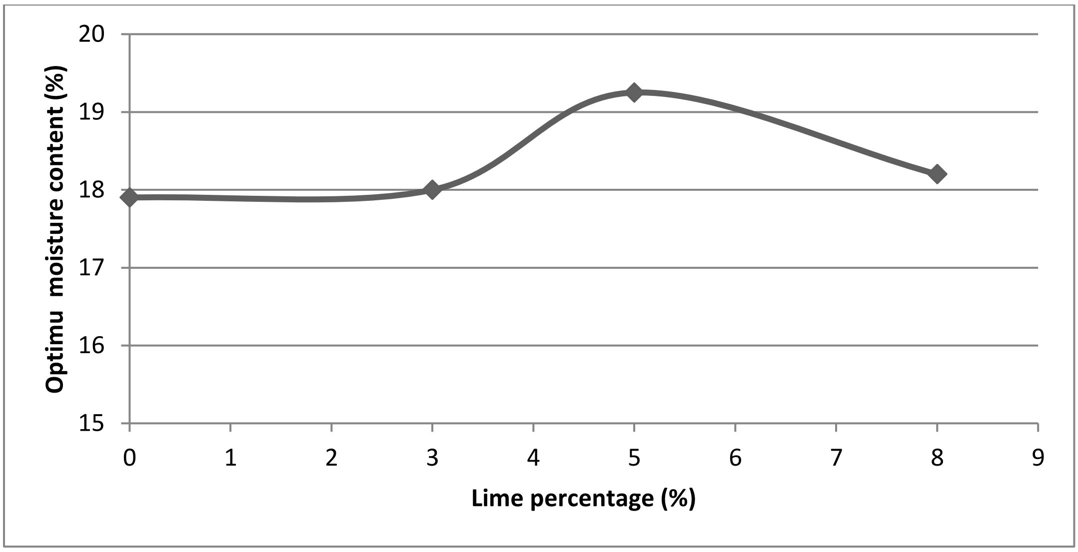


Figure 48: Change in OMC with lime percentage.

Maximum dry density and optimum moisture content with the fly ash percentage for soil stabilized with fly ash are shown in Figure 49 and Figure 50. It was observed that with 6% fly ash the MDD decreased slightly and then increased with the further addition of fly ash. OMC slightly increased with 6% fly ash and then declined slightly. The change can be considered negligible. Phanikumar (2009) observed an increase in MDD for an expansive soil stabilized with fly ash and it was concluded this behaviour is due to flocs formed rolling over them more easily during compaction.

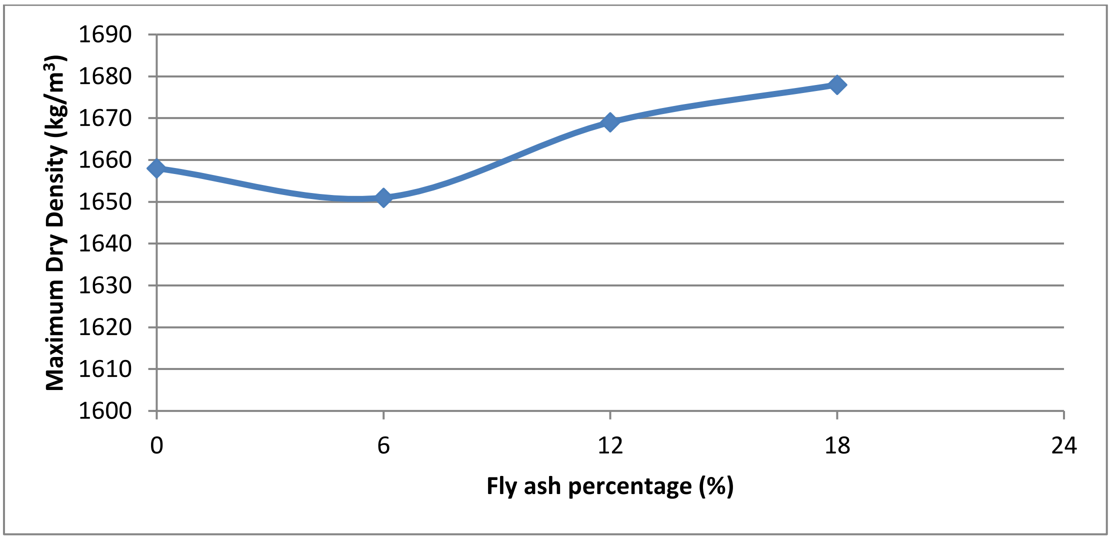


Figure 49: Change in MDD with fly ash percentage.

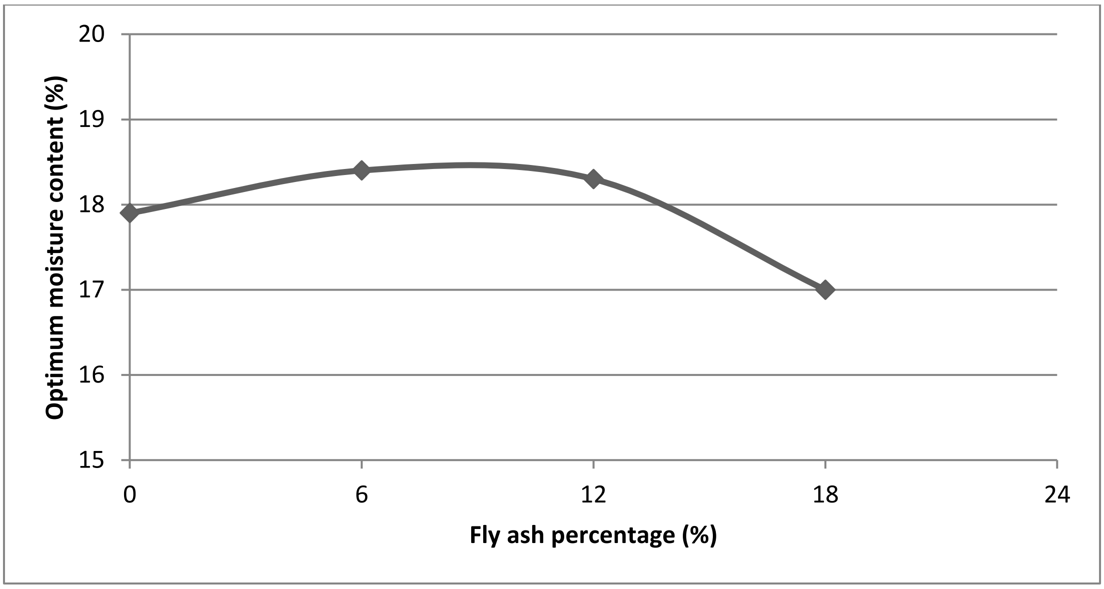


Figure 50: Change in OMC with the fly ash percentage.

Change in MDD and OMC with the fly ash percentage, for soil stabilized with 3% lime increasing fly ash percentage are shown in Figure 51 and Figure 52 respectively. It was observed that with the fly ash percentage the MDD decreased. For 18% fly ash with 3% lime, the decrease was about 3.3%. A similar trend in results was observed in (Zha et al., 2008) for soil stabilized with fly ash-lime admixture. The decrease is a result of flocculation and agglomeration, changing the gradation of soil which attributes for the increase in the void ratio.

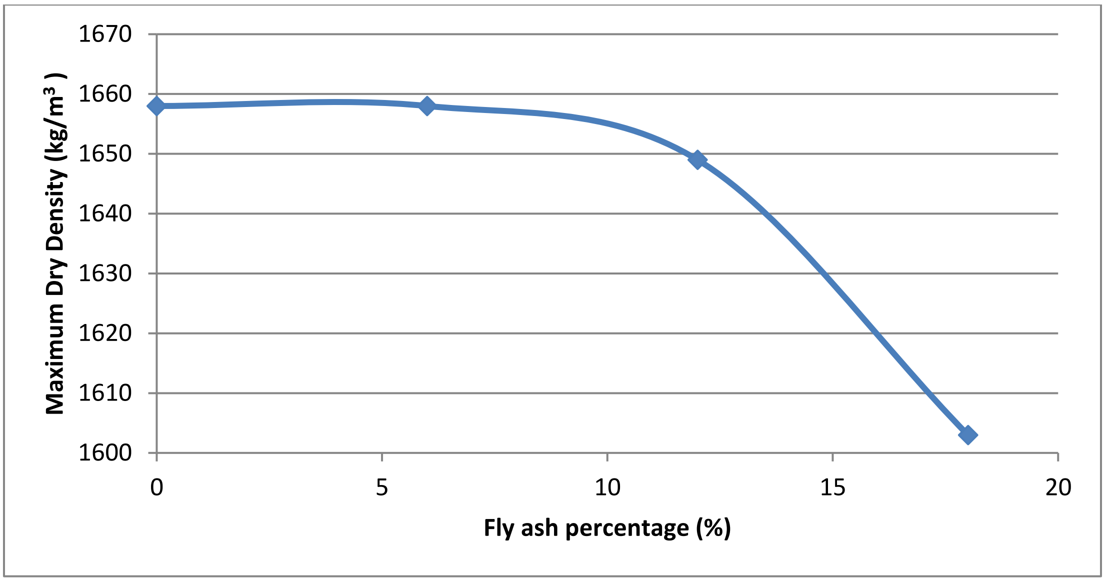


Figure 51: Change in MDD with fly ash percentage for lime-fly ash admixture.

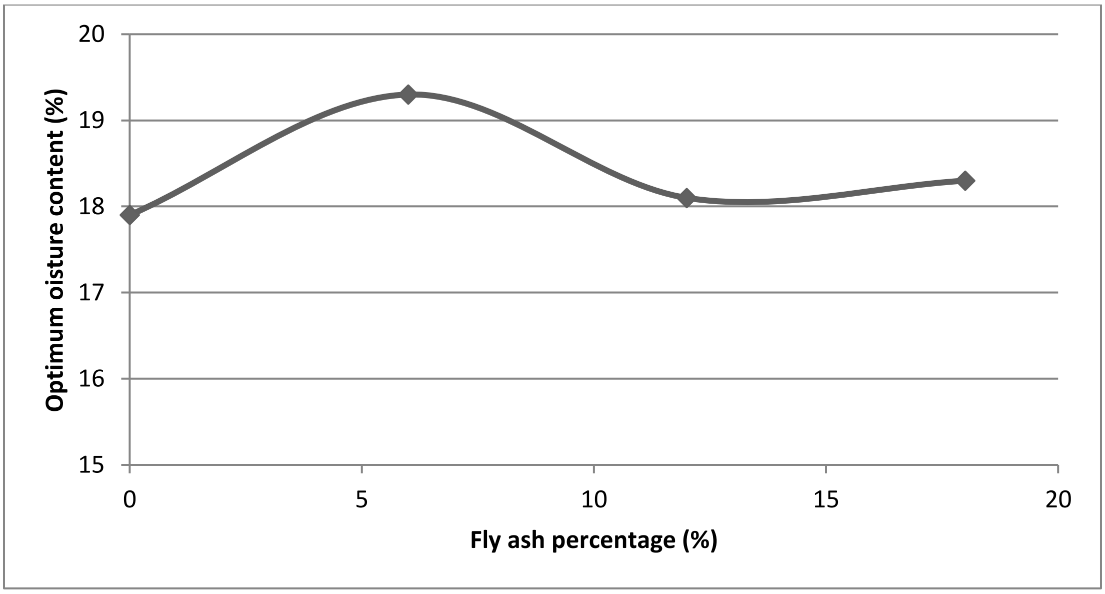


Figure 52: Change in OMC with fly ash for lime-fly ash admixture.

4.6.Unconfined compressive strength (UCS)

Figure 53illustrates the variation of UCS of lime stabilized soils with the lime percentage for 7, 14 and 28 days curing. With the increase of lime percentage up to 5%, the UCS of soil was increased and further addition reduced the UCS value slightly. With the curing time, the UCS of the soil increased compared to the untreated soil. For 28 days cured, 5% lime treated soil UCS increased by 2.4 times compared to the untreated soil. The variation of UCS with lime is not linear, similar to the results in Bell (1996) and Dash & Hussain (2012). Bell (1996) concludes that this behaviour is due to inadequate friction and cohesion in lime. Dash & Hussain (2012), attributed this behaviour is due to the excess formation of high porosity silica gel which reduces the strength gain through cementation.

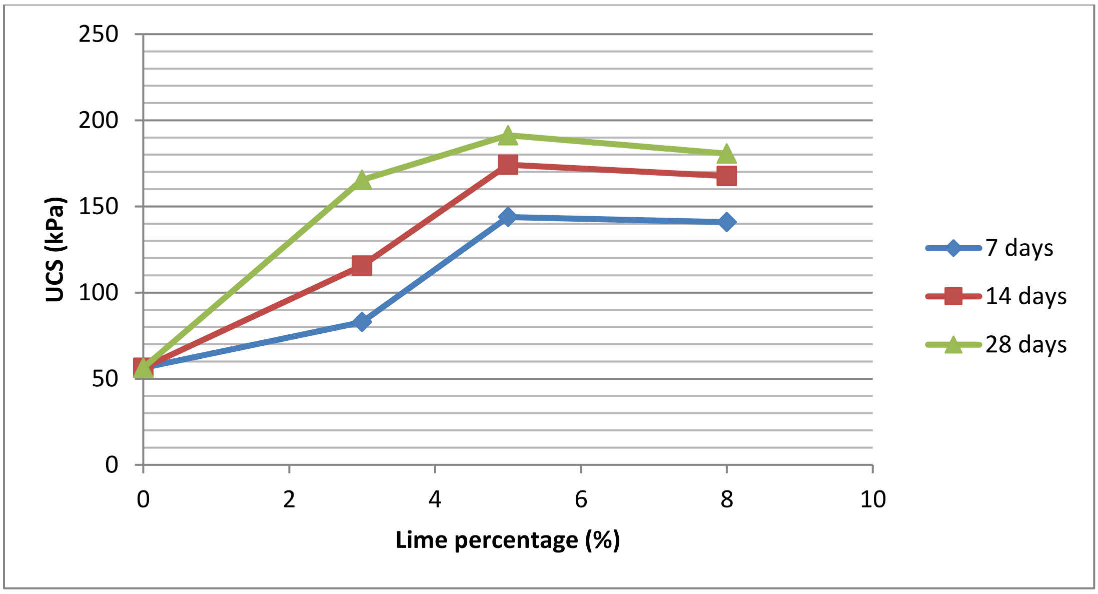


Figure 53: Effect of lime on UCS of lime stabilized soil

The variation of UCS with 6, 12 and 18% of fly ash by dry soil weight is shown in Figure 54. The increase in UCS for 28 days cured soil with 18% fly ash, compared to the untreated soil is 1.12 times. Even though there is an increase in UCS with fly ash percentage and with the curing time the increase is not significant as in lime stabilized soils. Strength gain in stabilized soils is mainly due to pozzolanic reactions. However, when class F fly ash is added to the soil alone since it has a low concentration of calcium ions to produce cementitious products the strength gain is not significant as in lime.

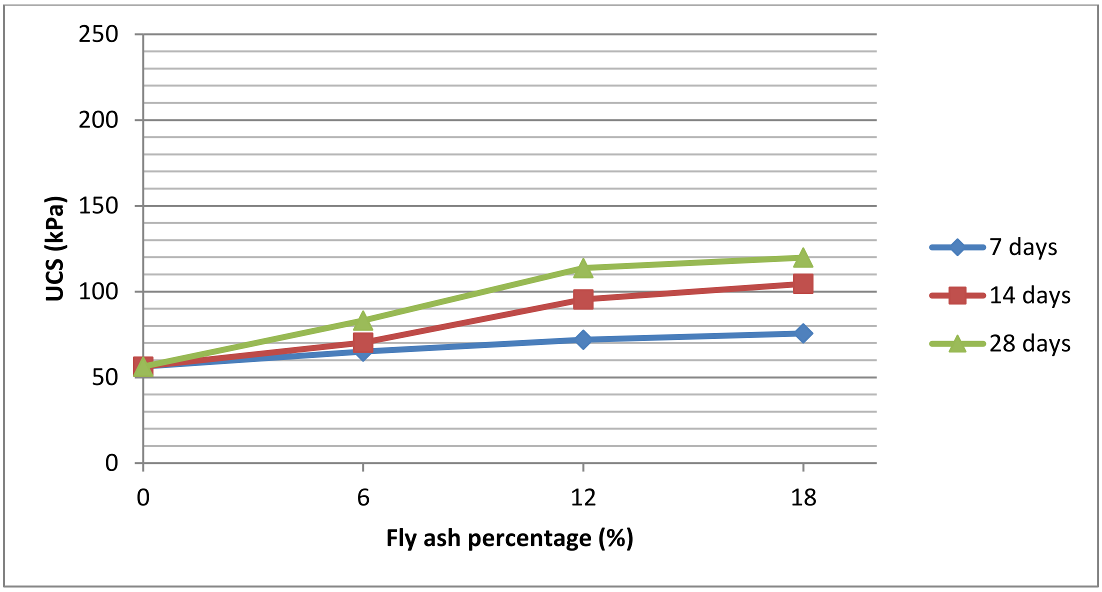


Figure 54: Effect of fly ash on the UCS of fly ash stabilized soil

Figure 55 illustrates the effect of lime-fly ash admixtures on the UCS of the soil. The gain in strength increases with fly ash percentage and curing time for soil treated with 3% constant lime and fly ash admixture. For 28 days cured, 3% lime and 12% fly ash treated sample, the UCS increased by about 2.6 times. Strength gain is not linear similar to lime stabilization, as UCS slightly declined with the addition of 18% fly ash cured for 14 days and 28 days. The reason is that when fly ash percentage increases the unbound fly ash particles act as silt particles which have neither appreciable friction nor cohesion (Zha et al., 2008). A decrease in the strength gaining rate with curing time was observed for lime-fly ash admixture treated soil. This behaviour may be attributed to limited calcium ions to react with silica and alumina to produce cementitious products.

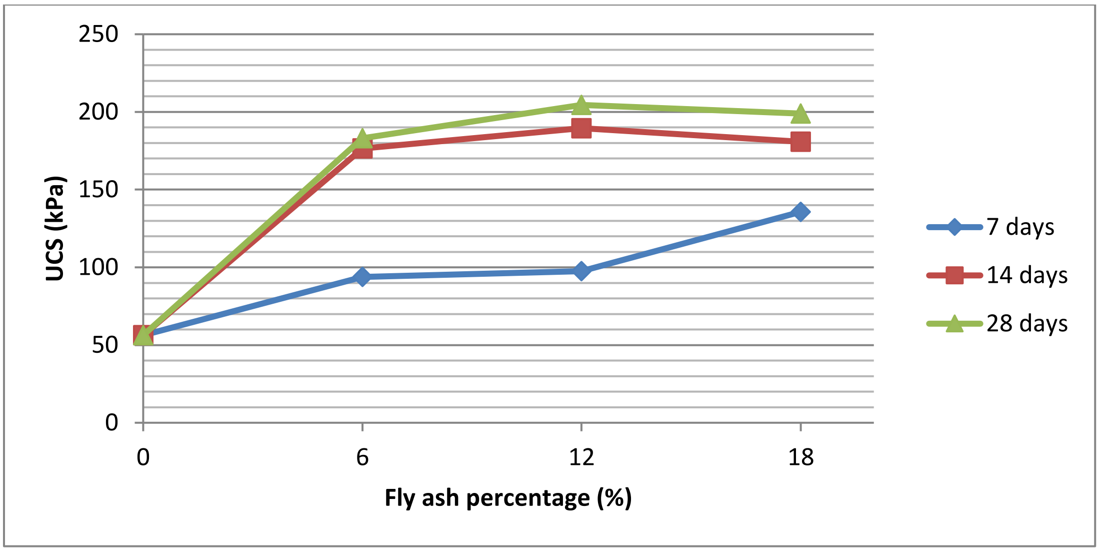


Figure 55: Effect of lime-fly ash admixtures on UCS of fly ash stabilized soil

4.7.California Bearing Ratio (CBR)

Change in four days soaked CBR value, for lime stabilized soil is shown in Figure 56. It was observed that with the lime percentage from 0 to 8% CBR value increased from 3.5% to 23.7%. A similar trend was observed in the studies of Bell (1996), for different clay minerals and Osinubi (1998), for a lateritic soil with lime stabilization.

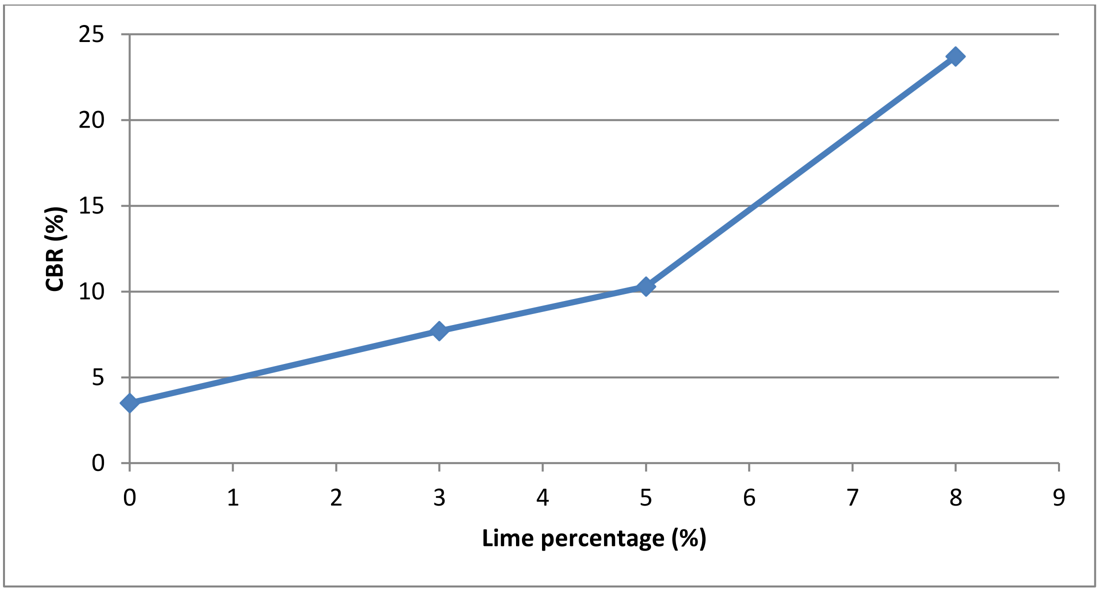


Figure 56: Change in CBR for lime stabilization.

Figure 57 illustrates the change in CBR value for fly ash addition from 0 to 18%. CBR value increased from 3.5% to 5.6% with the addition of 18% fly ash. The increasing rate of CBR is comparatively low when compared with CBR value for lime stabilized soil.

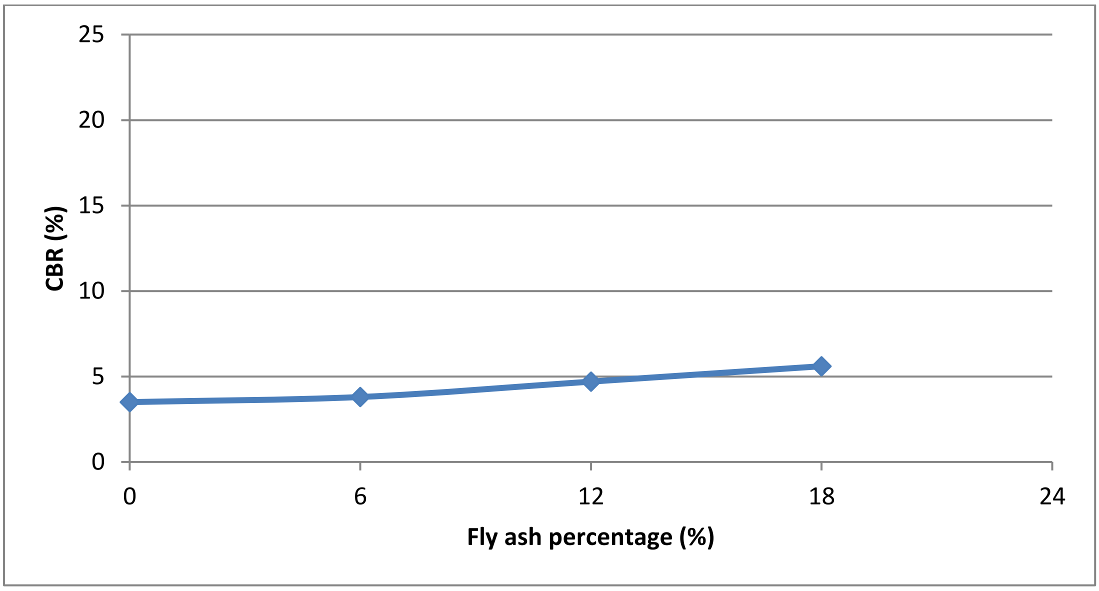


Figure 57: Change in CBR with fly ash percentage.

The CBR behaviour for soil stabilized with 3% lime increasing the fly ash percentage is shown in Figure 58. Similar to the previous results from lime stabilization and fly ash stabilization the CBR value for soil stabilized with lime-fly ash admixtures increased. With 18% fly ash the CBR value increased up to 15.2%. The rate of increase in CBR value for lime-fly ash stabilized soils is higher than soil stabilized with fly ash alone.

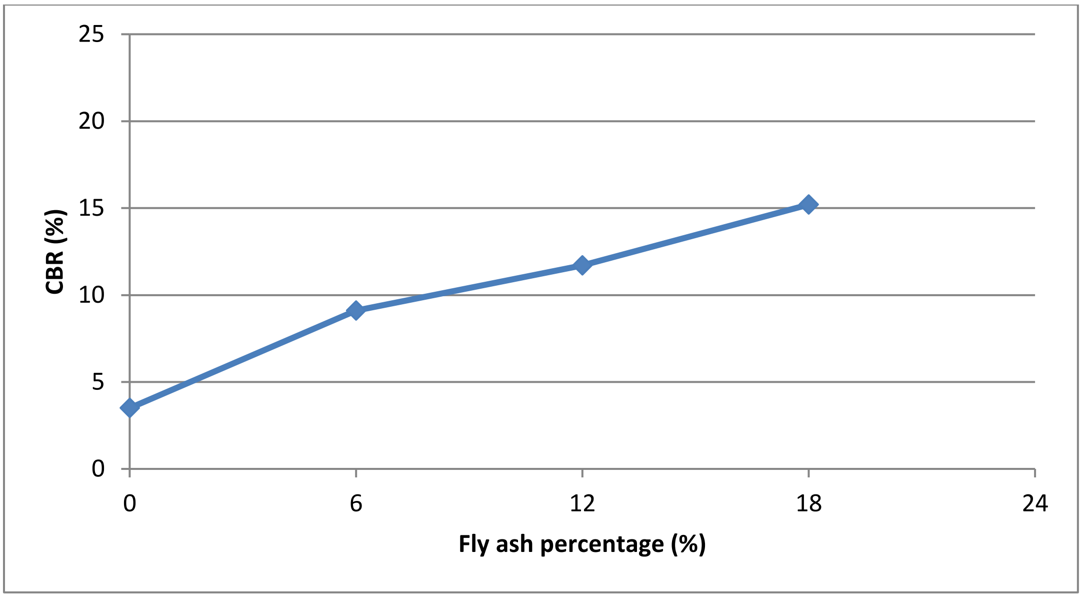


Figure 58: Change in CBR with lime-fly ash admixture percentage.

The residual soil used for the study has a CBR value of 3.5 where the subgrade strength class is S2 according to TRRL (1984). Therefore, granular road base with surface dressing, a capping of about 200 mm is required. To use the same residual soil improved with lime and fly ash as a capping layer CBR of more than 15 should be achieved. Using 6% lime or 3% lime with 18% fly ash, the required specifications can be achieved.

5. CONCLUSION

This thesis is focused on investigating the improvement of an unsuitable residual soil using lime, fly ash and lime-fly ash admixture stabilization. The study was carried out to identify the variation of some important parameters such as liquid limit, plastic limit, plasticity index, compaction characteristics, unconfined compressive strength and California bearing ratio with different additives. Furthermore, the suitability of the stabilized soil with different mix proportions of lime and fly ash to be used in roads, as capping or imported subgrade material was studied.

Soil treated with lime showed a slight increase in LL with the lime percentage. PL increased with 3% lime and then the variation of Pl with further addition of lime was not that significant. Due to these variations, PI showed a decline with 3% lime and then remained more or less steady. The decrease in PI attributed to cation exchange and flocculation of clay minerals in the soil.

With fly ash stabilization, no significant variation in plasticity characteristics was observed. Low calcium (Ca^{2+}) percentage in fly ash is not enough for the immediate reactions to take place, changing the gradation of the soil.

Stabilizing soil with lime-fly ash admixtures increased the LL as well as PL with 6% fly ash and then remained more or less constant with the increase in additive percentage. Overall, no significant change in PI was observed. The increase in LL may be due to formation of water holding gelatinous products due to the presence of reactive silica in soil as well as from fly ash.

For lime as well as lime-fly ash admixtures a decrease in MDD was observed. The decline is due to flocculation of particles increasing the void ratio and the low specific gravity of lime and fly ash. With fly ash alone, a slight increase in MDD was observed as flocs formed can roll over them more easily and voids are filled with fly ash during compaction.

No significant variation was observed in OMC for all the cases as the variation in MDD is marginal (maximum variation in MDD is 3.3%).

An increase in UCS of stabilized soil with the additive percentage and curing time was observed. However, the increase is not linear for lime and lime-fly ash admixtures. The optimum lime content from UCS is 5% lime by dry soil weight. For 5% lime, a strength gain of 2.4 times the UCS of untreated soil was observed after curing for 28 days. For 3% lime the optimum fly ash percentage is 12% as for the UCS results. With 3% lime and 12% fly ash a strength gain of 2.6 times the untreated soil was observed. Strength gain in fly ash treated soil is not significant as lime or lime-fly ash admixture treated soil.

The increase in UCS of soil treated with different mix proportions is due to the pozzolanic reactions. With the additive percentages and time, amount of cementitious products formed increases and strength gain increases. The decrease in UCS beyond optimum additive percentage is due to excess lime and fly ash act as silts which has neither appreciable friction nor cohesion.

4 days soaked CBR value of lime stabilized soils increased by about 20% with 8% lime. To use the soil as a road capping layer material a lime percentage of around 6% is adequate. Treating the soil with fly ash has no significant effect on the CBR value similar to the other reactions. 18% fly ash with 3% lime can achieve the required CBR value for stabilizing residual soils as capping layer materials.

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